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Evaluation of Arkansas River Design Criteria for Stabilization and Rectification of the Channel

By Margaret S. Petersen, Emmett M. Laursen

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December 1986

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This report covers work under part one of a two-part study for the Vicksburg District as part of their design studies for stabilization and canalization of the Lower Red River, Louisiana. Studies summarized in this report are limited to evaluation of the effectiveness of criteria used for similar stabilization work in providing navigable depths in the Lower Arkansas River, Arkansas, a similar project that has been in operation since 1969.

Studies indicate that criteria used for stabilization in Arkansas River Pools 9 through 3 have been very successful in providing design depths, and that required maintenance dredging in the period 1978-1984 has been negligible, averaging about 150,000 cubic yards per year. Studies further indicated that deposition problems in Pool 2 (where maintenance dredging had averaged 430,000 cubic yards per year in the 1978-1984 period)
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probably are more related to pool characteristics than to design of stabilization work. Pool 2 is significantly longer and has more storage at normal pool level than Pools 9-3, and it is subject to open-river flow conditions more rarely than the other pools (spillway gates fully open about once in seven years, on the average, compared to annually at the other pools.

PREFACE

The work described in this report was performed during the period January to December 1986 under Contract DACW39-86-C-0027, titled, "Evaluation of Channel Design Criteria - Arkansas River, Arkansas," between the U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, and The University of Arizona, College of Engineering and Mines, Department of Civil Engineering and Engineering Mechanics, Tucson, Arizona. The research was sponsored by the Hydraulics Laboratory (HL), WES, for the U.S. Army Engineer District, Vicksburg (LMK).

This report describes part one of a two-part study to evaluate the effectiveness of design criteria used for stabilization and rectification of the Arkansas River Navigation Project, Arkansas, and to modify those criteria, as appropriate, for application in a similar project on the Red River in Louisiana. Part one is limited to evaluation of the Arkansas River work and to development of an understanding of the basic river processes involved in adjustment of Arkansas River morphology to project conditions.

The study was performed and the report written by Professor Margaret S. Petersen and Dr. Emmett M. Laursen. Graduate assistants for the study were Amgad S. E. Elansary, Seree Chanyotha and Steve M. Cooke.

The contract was monitored by Mr. Robert W. McCarley, who served as the Contracting Officer's Representative, under the general supervision of Mr. Marden B. Boyd, Chief of the Hydraulic Analysis Division, HL, and Mr. William A. Thomas of the Hydraulic Analysis Division. Messrs. Phil G. Combs, LMKED-HH, Max S. Lamb, Lower Mississippi Valley Division (LMVED-RP), and Tasso Schmidgall, Southwestern Division (SWDED-WA) reviewed the report in detail and provided valuable comments. Messrs. Jack Woolfolk and Gist Wilbur and Ms. Margaret Rohan of the Little Rock District provided much of the necessary information and data on the Arkansas River for study and evaluation.

COL Allen F. Grum, USA, was the previous Director of WES. COL Dwayne G. Lee, CE, is the present Commander and Director. Dr. Robert W. Whalin is Technical Director. COL Patrick M. Stevens IV is the Vicksburg District Commander.

EVALUATION OF ARKANSAS RIVER DESIGN CRITERIA FOR

STABILIZATION AND RECTIFICATION OF THE CHANNEL

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EVALUATION OF ARKANSAS RIVER DESIGN CRITERIA FOR STABILIZATION AND RECTIFICATION OF THE CHANNEL

CHAPTER 1 - INTRODUCTION

1-1 OBJECTIVES OF STUDY

Objectives of the research program funded by the U.S.A.E. Waterways Experiment Station in January 1986, at The University of Arizona, Department of Civil Engineering and Engineering Mechanics were as follows:

1. Evaluate the effectiveness of design criteria used for stabilization and rectification of the Arkansas River Navigation Project since the project became operational in the late 1960's.

2. Evaluate the applicability of those criteria to the Red River Navigation Project by conducting an analysis of the hydrology, geomorphology, planned layout of the navigation channel, and hydraulic design of navigation structures for the Red River.

This report covers Item 1. Item 2 work will be conducted in the future when approved and funded.

Work reported on herein included evaluating the Arkansas River rectified alignment, control structures, and design channel widths and depths; study of surveyed cross sections, water surface profiles, maintenance dredging, and additional contraction works added to maintain navigation depth during the time the project has been in operation; and examination of the basic river processes involved in adjustment of Arkansas River morphology to project conditions. Arkansas River design criteria are summarized, and present characteristics of Pools 9 through 2 downstream of Dardanelle Dam are described, with particular attention given to problem

areas requiring significant maintenance dredging. Terms having specialized meaning in river engineering are defined in Appendix A, Glossary.

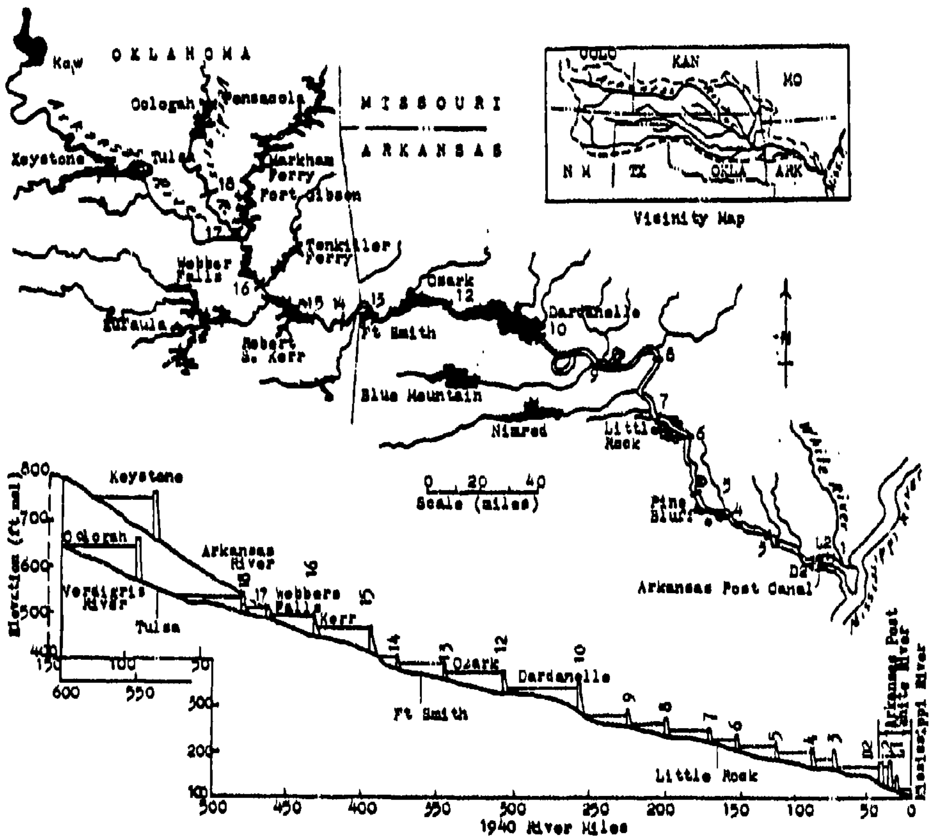
1-2 STUDY AREA

Locations of components of the Arkansas River waterway, including the low-lift navigation locks and dams, are shown in Figure 1-1. The natural sediment load and flow regime of the lower Arkansas River have been modified significantly by construction of major storage dams on the main stem and tributaries in Oklahoma. Additionally, sediment is trapped in the Ozark and Dardanelle pools in Western Arkansas, Figure 1-1. The suspended sediment load contributed by tributaries in Oklahoma is indicated schematically by band width in Figure 1-2. Tributary inflow in Arkansas is minor, with the only significant inflows contributed by the Petit Jean River and Fourche Le Fave River that enter the Arkansas along the south bank downstream of Dardanelle Dam, Figure 1-2.

The canalized Arkansas River reach downstream of Dardanelle Dam most closely resembles the Red River navigation project reach downstream of Shreveport. Accordingly, current work was largely limited to evaluation of Arkansas River criteria downstream of Dardanelle Dam.

1-3 STUDY LIMITATIONS

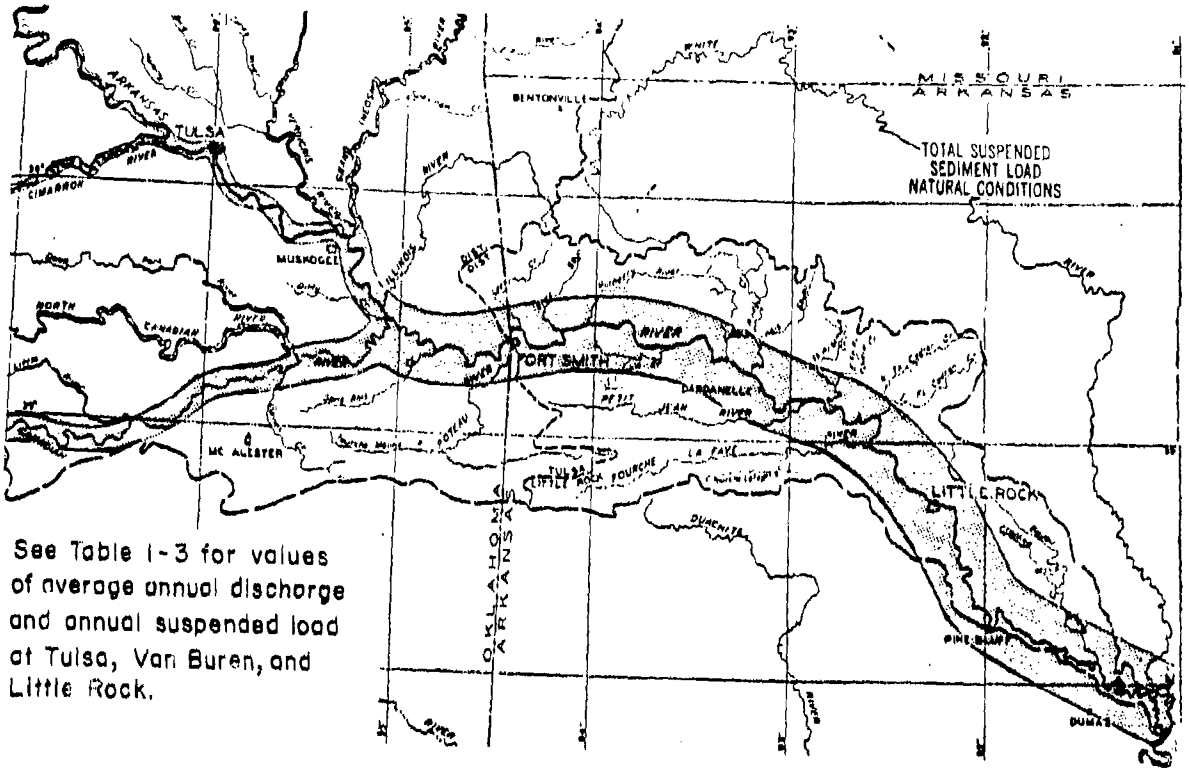
Archives of the Little Rock District contained much data from the original design studies for the Arkansas River project; however, field data under project conditions in the study area are somewhat limited. Recent field data included the following:



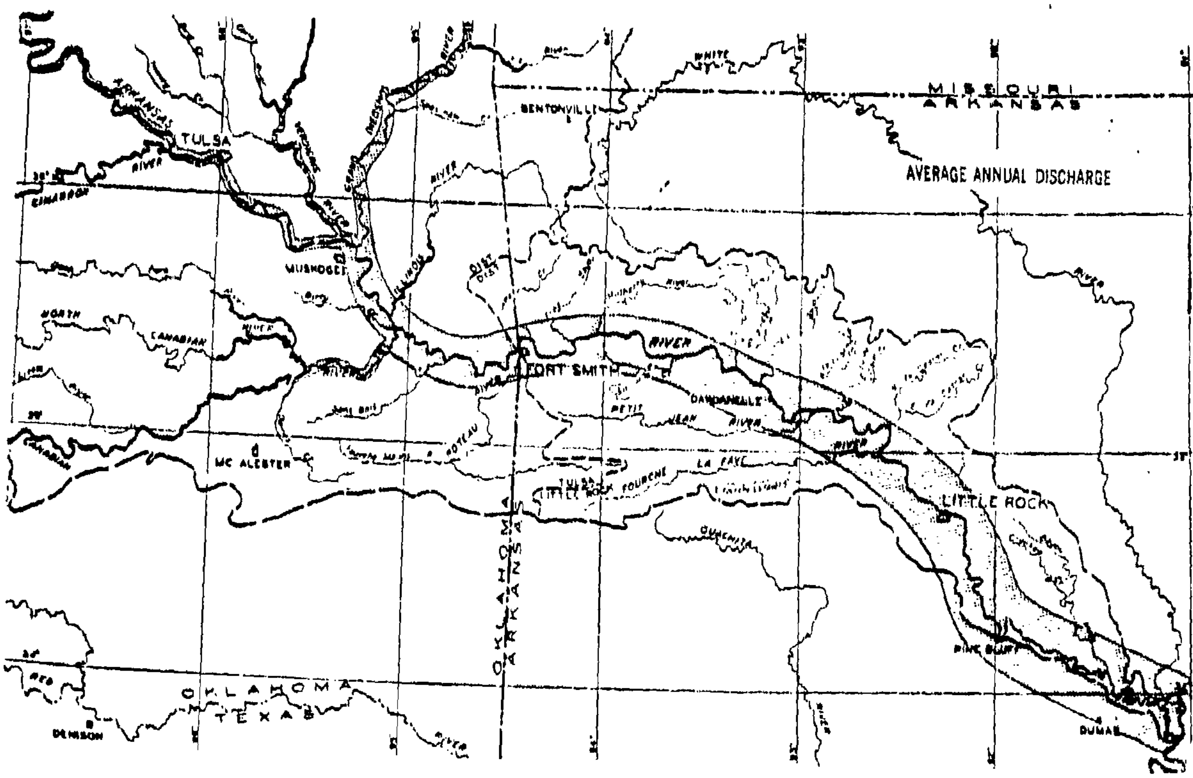
Arkansas River Project.

Arkansas River Waterway
Plan and Profile

Figure 1-1



See Table 1-3 for values of average annual discharge and annual suspended load at Tulsa, Van Buren, and Little Rock.



Arkansas River

Suspended Sediment Load and Average Annual Discharge

Figure 1-2

- Surveyed cross sections in various years, 1969-1981 (not all pools were surveyed in all years)

- Measurements of bed material size composition in various years, 1970-1976

- Measurements of suspended sediment load in various years

- Computed daily sediment transport at Van Buren, Ozark, Dardanelle, and Little Rock under project conditions

- Aerial mosaics flown in 1984 showing stabilization and rectification structures

- Records of maintenance dredging

- Limited water surface profile data

Water surface profiles subsequent to 1978, additional information on bed material composition over time, additional measurements of sediment load, and more recent cross section data would have been helpful in this study; however, available information was generally adequate.

1-4 ARKANSAS RIVER PROJECT

The Arkansas River Navigation Project was authorized by Congress in the River and Harbor Act of July 1946 (later amended by the Flood Control Acts of 1948 and 1950) for development of navigation, flood control, hydroelectric power, and other purposes, but no funds were appropriated for the project until 1949. The navigation channel has a minimum depth of 9 ft with 17 locks and dams (12 in Arkansas and 5 in Oklahoma) to enable navigating the 420-ft rise in elevation from the Mississippi River to the vicinity of Tulsa, Oklahoma. In addition, power facilities were installed at four of the navigation locks and dams (including Dardanelle and Ozark Dams in Arkansas); additional power facilities are now (1986) under study by local entities. Also, eight large multiple-purpose reservoirs were

Table 1-1

RESERVOIR PROJECTS

Project	Stream	River Mile*	Primary Purposes	Controlled Storage (ac-ft): Sediment & Dead Storage	Controlled Storage (ac-ft): Total
Kaw	Arkansas	654.0	Flood-control, water supply for municipal use, and sediment control	85,100	1,348,000
Keystone	Arkansas	538.8	Flood-control, water supply for municipal use, future power, navigation, and sediment control	415,000	1,879,000
Oologah	Verdigris	90.2	Flood-control, water supply for municipal use, future power, and navigation	9,300	1,021,000
Fort Gibson	Grand (Neosho)	7.7	Flood-control and power	311,000	1,287,000
Webbers Falls	Arkansas	432.3	Navigation and power	164,000	164,000
Tenkiller Ferry	Illinois	12.8	Flood-control and power	285,000	1,230,000
Eufaula	Candian	27.0	Flood-control, power, and sediment control	897,000	3,848,000
Robert S. Kerr	Arkansas	395.4	Navigation and power	380,000	500,000
Ózark	Arkansas	319.0	Navigation and power	92,000	92,000
Dardanelle	Arkansas	257.8	Navigation and power	70,000	500,000

*1940 Milepost (see Glossary).

constructed in Eastern Oklahoma, Figure 1-1 and Table 1-1. Pertinent data for the navigation locks and dams in Arkansas are listed in Table 1-2.

The project was delayed by the Korean War in the early 1950's, and it was not until 1956 that construction was initiated on Keystone and Eufaula Reservoirs in Oklahoma. Those reservoirs significantly decreased the natural sediment load of the Arkansas River prior to construction of Dardanelle Dam begun in 1959. Construction of Lock and Dam 1 and 2 was initiated in 1963, and construction of the navigation structures proceeded upstream. By 1968, all the navigation locks and dams in Arkansas and Oklahoma were under construction. The project was completed to Little Rock in December 1966, to Fort Smith in December 1969 and to Catoosa-Tulsa in December 1970.

Hydraulic Characteristics, Preproject Conditions - In its natural state, the Arkansas River carried a very large sediment load, with a mean sediment content by weight of about 0.3 percent. The suspended total loads of the Arkansas River and Red River are compared to those of other rivers in the United States in Table 1-3. The preproject sediment load and discharge of the Arkansas River and major tributaries are shown schematically on Figure 1-2.

At Van Buren, near the Arkansas-Oklahoma border, natural average flow was 36,100 cfs, and the annual sediment load was 97.2 million tons, or 2,690 tons per cfs. Between Van Buren and Little Rock, Arkansas (188 miles) there was relatively little sediment production. At Little Rock, the average annual discharge was 47,500 cfs, and the annual sediment load was 104.9 million tons, or 2,200 tons per cfs. The river channel was about 2,000 ft

Table 1-2
Pertinent Data
Locks and Dams in Arkansas

LOCK AND DAM NUMBER		1 NORRELL	2	3	4	5	6 DAVID D. TERRY	7 MURRAY	8 TOAD SUCK FERRY	9	10 DARDANELLE	12 OZARK	13
GENERAL	PURPOSE	NAVIGATION	NAVIGATION	NAVIGATION	NAVIGATION	NAVIGATION	NAVIGATION	NAVIGATION	NAVIGATION	NAVIGATION	POWER AND NAVIGATION	POWER NAVIGATION RECREATION [illegible]	POWER STORAGE AND NAVIGATION
GENERAL	LOCATION: Navigation Mile	10.3	13.3 (Lock)	50.2	66.0	86.3	108.1	125.4	155.9	176.9	205.5	256.8	292.8
GENERAL	LOCATION: River Mile	—	40.8 (Dam)	72.5	87.7		184.4	172.0	201.2	234.9	267.8	300.9	346.5
SPILLWAY	GATES, number and size	UNGATED WEIR	16-60x30	12-60x25	5-35x25 / 6-60x25	13-30x30	17-60x27	14-60x33	16-60x24	14-60x35	20-50x30	15-50x46	15-60x30
SPILLWAY	PIER WIDTH	NONE	10	10	10	10	10	10	10	10	10	10	10
SPILLWAY	TOTAL LENGTH (incl. one end pier)	277	1,180	1,200	1,100	1,030	1,190	960	1,120	940	1,210	900	1,030
SPILLWAY	ELEVATION OF CREST (m.s.l.)	142	134	150	D Dam-160 / B Dam-174	165	206	210	242	253	300	327	363
SPILLWAY	FULLY OPEN GATE LIP El (m.s.l.)	N/A	180	207	217	248	252	288	294	313.5	341	363	414
EMBANKMENT	RIGHT BANK LENGTH		6983	900	780	665	2050	210	90	NONE	NONE	1530	463
EMBANKMENT	RIGHT BANK CREST ELEVATION	182	[illegible]	Nav Pool 185 / 186	206-214	223	236	259	260	N/A	N/A	382	402
EMBANKMENT	LEFT BANK LENGTH	6400	32,000	3870	3000	5700	4050	2,350	300	825	895	230	3210
EMBANKMENT	LEFT BANK CREST ELEVATION	170.0	190	192	Nav Pool-197 / 199	223	241	252	279	292	336	382	395
EMBANKMENT	OVERFLOW SECTION LOCATION & LENGTH	Right Bank	Right Bank 1800	Right Bank 985	L. Bank 5500 / R. Bank 780	L. Bank 5700 / R. Bank 625	Right Bank 2050	Left Bank 2,550	Right Bank 80	L. Bank-825 Crest-292	NONE	NONE	Left Bank 3210
LOCK: ELEVATIONS	UPPER POOL NORMAL	142.0	162	182	196	213	231	249	265	284-287	338	372	392 (Top Pwr Pool)
LOCK: ELEVATIONS	UPPER POOL MINIMUM	142.0	157	177	191	208	226	244	260	279	336	370	391
LOCK: ELEVATIONS	UPPER POOL (max. navigation)	NONE	172.0	194.4	203.5	222.0	240.9	256.4	276.4	290.3	338	372	396.5
LOCK: ELEVATIONS	LOWER POOL (normal) (no flow)	(Low) 112.0 / (Normal) 130.0	142	162	182	196	213	231	249	265	284	336	372
LOCK: ELEVATIONS	LOWER POOL (max. navigation condition)	NONE	155	196.4	202.2	221.4	239.7	255.2	275.2	289.1	313.6	362	395.7
LOCK: ELEVATIONS	UPPER SILL	126	144	164	178	195	213	231	247	266	321	354	374
LOCK: ELEVATIONS	LOWER SILL	97	128	140	168	182	199	217	233	251	270	321	356
LOCK: ELEVATIONS	CHAMBER FLOOR	95	127	147	165	179	196	200	Varies 210-231	247	263	Varies 305-319	352
LOCK: DIMENSIONS (in feet)	CHAMBER WIDTH	110	110	110	110	110	110	110	110	110	110	110	110
LOCK: DIMENSIONS (in feet)	CHAMBER LENGTH (usable)	600	600	600	600	600	600	600	600	600	600	600	600
LOCK: DIMENSIONS (in feet)	CHAMBER LENGTH (between pintles)	670	670	670	670	670	670	670	670	670	670	670	670
LOCK: DIMENSIONS (in feet)	LIFT NORMAL	30	20	20	14	17	18	18	16	19	54	34	20
LOCK: DIMENSIONS (in feet)	DEPTH ABOVE LOWER SILL (min.)	15	14	14	14	14	14	14	14	14	14	15	14
LOCK: DIMENSIONS (in feet)	HEIGHT OF UPPER GATE	25	33	20	23	20	28	25	30	25	21	25	25
LOCK: DIMENSIONS (in feet)	HEIGHT OF LOWER GATE	52	44	44	38	40	42	40	42	40	73	57	47
LOCK: DIMENSIONS (in feet)	MIN. CUSHION DEPTH in CHAMBER	17	12	15	17	17	17	29	17	18	21-24	19-35	17
LOCK: DIMENSIONS (in feet)	CULVERT SIZE	12x12	12x12	12x12	12x12	12x12	12x12	12x12	12x12	12x12	14.5x13.0	12x12	12x12
FOUNDATIONS: TYPE		Conc piles in sand	Conc piling in sand	Steel piling in sand	Concrete piling in sand	Tertiary [illegible] clay	Steel piles in sand	Shale	Shale and sandstone	Shale	[illegible] sandstone	Sandstone	Shale and sandstone
FOUNDATIONS: TYPE STRUCTURE	GUIDE WALLS	U.S.-Cellular / D.S.-Floating	Cellular	Cellular	Cellular	Conc Gravity	Cellular	Concrete Gravity	Concrete Gravity	Concrete Gravity	U.S. Cellular / D.S. [illegible]	Concrete Gravity	Concrete Gravity
FOUNDATIONS: TYPE STRUCTURE	GUARD WALLS	Concrete semi-gravity	Concrete semi-gravity	Concrete semi-gravity	Concrete semi-gravity	Concrete gravity	Concrete semi-gravity	Concrete gravity	Concrete gravity	Concrete gravity	Concrete gravity	Concrete gravity	Concrete gravity
FOUNDATIONS: TYPE STRUCTURE	GATE BAY WALLS	Concrete semi-gravity	Concrete semi-gravity	Concrete semi-gravity	Concrete semi-gravity	Concrete [illegible]	Concrete semi-gravity	Concrete gravity	Concrete gravity	Concrete gravity	Concrete gravity	Concrete gravity	Concrete gravity
FOUNDATIONS: TYPE STRUCTURE	CHAMBER WALLS	Concrete semi-gravity	Concrete semi-gravity	Concrete semi-gravity	Concrete semi-gravity	Concrete [illegible]	Concrete semi-gravity	Concrete gravity	Concrete gravity	Concrete gravity	Concrete gravity	Concrete gravity	Concrete gravity

Table 1-3

Suspended Sediment and Flow Data

for Rivers in the United States

River	Station	Drainage area (sq. mi.)	Average flow (c.f.s.)	Mean sediment content (percent by weight)	Annual suspended sediment load: Tons	Annual suspended sediment load: Acre-feet (1)
Arkansas	Tulsa, Okla.	74,615	7,700	0.31	23,400,000	17,900
Arkansas	Van Buren, Ark.	150,483	36,100	0.27	97,200,000	74,400
Arkansas	Little Rock, Ark.	158,201	47,500	0.22	104,900,000	80,300
Verdigris	Near mouth, Okla.	8,303	4,900	0.13	6,300,000	4,800
Grand (Neosho)	Near mouth, Okla.	12,492	10,000	0.10	10,000,000	7,700
Illinois	Tenkiller Ferry	1,610	1,800	0.05	800,000	600
Canadian	Whitefield, Okla.	47,576	6,700	0.75	49,700,000	38,000
White	Clarendon, Ark.	25,497	30,000	0.04	11,000,000	8,400
Missouri	Omaha, Nebr.	322,800	25,100	0.64	158,000,000	120,900
Upper Mississippi	Burlington, Iowa	114,000	57,000	0.03	14,300,000	10,900
Ohio	Paducah, Ky.	203,000	174,000	0.03	58,800,000	45,000
Tennessee	Johnsonville, Tenn.	38,520	64,500	0.03	18,400,000	14,100
Red River	Gainesville, Tex.	(2)30,782	3,350	0.52	17,000,000	13,000
Red	Shreveport, La.	60,600	24,800	0.13	32,000,000	24,500
Red	Alexandria, La.	67,333	31,000	0.12	37,000,000	28,300
Rio Grande	Elephant Butte	25,923	1,500	1.09	16,100,000	12,300
Colorado	Grand Canyon Station	137,800	17,700	0.98	172,000,000	131,600
Colorado (Texas)	San Saba, Tex.	(3)30,600	1,500	0.67	10,000,000	7,700

(1) Weight sediment assumed as 60#/cu. ft.
(2) 24,846 square miles contribute runoff.
(3) 18,800 square miles contribute runoff.

wide, and average slope was about 0.8 ft per mile. Downstream from Little Rock to Pine Bluff (55 miles) the slope steepened somewhat, to about 0.86 ft per mile, and from Pine Bluff to the confluence of the Arkansas with the Mississippi (110 miles) river width was about 2,000 feet, and the slope about 0.7 ft per mile. There are no major tributaries below Little Rock.

The meander belt of the Arkansas River varies in width, but is generally from one to five miles wide. There are a number of rock outcrops and stable points along the river in Arkansas, and prior to stabilization the river meandered between them. Alluvium in the river bed above thalweg level was naturally about 70 percent sand (2.0 to 0.004 mm) and 30 percent silt and clay (<0.004 mm). Downstream to Little Rock the river bed was composed of sands and gravels from a few feet to 30 feet deep over underlying rock; downstream of Little Rock bedrock is at great depth.

The predominant size of bed material was medium sand (0.25 to 0.50 mm), with the material generally becoming finer in a downstream direction and coarser with depth below the river bed. Some gravel was mixed with sand throughout the alluvial bed and was found in greatest concentration near the bed rock. Gravel was primarily visible on the river bed near the upstream ends of sand bars. For design of the Arkansas River navigation project, 286 bed load samples were taken at Van Buren, Dardanelle, and Little Rock. The maximum sizes of gravels trapped with discharges up to 650,000 cfs ranged from about 90 mm (3.5 inches) at Van Buren to 50 mm (2 inches) at Little Rock.

At Van Buren, the discharge had ranged from 850,000 to 300 cfs, and average annual flow ranged from 75,900 to 6,370 cfs (55 to 4.6 million

acre feet per year) prior to construction of the upstream storage reservoirs. Maximum and minimum sediment loads at Van Buren were estimated to be 137.7 and 9.35 million tons per year, respectively. About half of the total load was carried by high flows experienced less than 10 percent of the time.

Under natural conditions about 25 percent of the suspended load of the lower Arkansas River was sand, with very fine sand (0.062 to 0.125 mm) predominating. The remaining suspended load was silts and clays. The bed load was estimated to be somewhat less than 10 percent of the total load and was predominantly sand and gravel.

Estimated Project Modification of Hydraulic Characteristics - Large storage reservoirs in the Upper Arkansas River Basin in Oklahoma, listed in Table 1-1 and shown in Figure 1-1, have modified the sediment regime of the river significantly. Essentially all the silt-clay load (termed wash load in the Arkansas River studies) was expected to be trapped in the upstream storage reservoirs, and it was expected that under project conditions the wash load in the lower river would be derived from tributary inflows and from bed and bank erosion. Wash load is not closely related to discharge, but is indirectly related through rainfall washing eroded material into the streams and through bank caving. Studies during project design indicated it was unlikely that any of the clay load would be deposited in the navigation system.

To limit the sediment load moving into the lower Arkansas River navigation reach, the upstream storage reservoirs were closed prior to or shortly after closure of Dardanelle Dam in 1963. Studies indicated that

sediment deposition in Dardanelle Reservoir would reduce reservoir storage by 38 percent in 50 years and that contraction works would be required eventually throughout the Dardanelle pool to maintain a navigation channel. Available information indicates that storage decreased about eight percent between 1964 and 1981. Additional contraction structures (extending for about three miles downstream from initial work) were constructed in 1973 primarily to realign the navigation channel to provide a better approach to a proposed bridge crossing.

Under preproject conditions the suspended sediment load at Dardanelle averaged 100.4 million tons per year; this was estimated to be reduced to 16 million tons per year under project conditions with the upstream medium-lift navigation dams (Webbers Falls, Robert S. Kerr, and Ozark) on the Arkansas River closed and the Oklahoma storage reservoirs in operation. In the 13-year period 1965-1977, with most of the upstream reservoirs in operation, the average annual suspended sediment inflow into Dardanelle Reservoir was about 8 million tons per year. Suspended sediment outflow from Dardanelle Reservoir averaged about 3.5 million tons per year over the 18-year period 1964-1981. Sediment transport under project conditions is discussed further in Chapter 5.

In design studies, the sand load at Dardanelle was projected to decrease as source material from the riverbed upstream coarsened, but the silt-clay load was projected to increase over time as storage in the medium-lift navigation dams upstream of Dardanelle became depleted by deposition and more fine material passed through the pools.

It was estimated that about 60 percent of the sand load entering Dardanelle Reservoir would be deposited in the pool, but that 90 percent of the silt-clay load would pass through. With the downstream banks stabilized, this reduction in wash load was expected to be maintained throughout the lower river downstream of Dardanelle. The wash load was considered to be too fine to be deposited in any of the downstream low-lift pools. The river was, however, projected to pick up an appreciable part of its normal sand load by the time it reached Pine Bluff. (No post-project data were available on size distribution of the suspended load downstream of Dardanelle.)

Stabilization and Rectification Work - The low-lift navigation dams were sited with the objective of limiting dredging to the heads of the pools, and special contraction works were designed for reaches immediately downstream of the dams and locks to aid in providing suitable depths and slopes so as to minimize loss of sediment transport capacity immediately below the structures. It was, however, considered essential that contraction works not raise water-surface elevations significantly and reduce preproject flood-carrying capacity of the river.

In its natural state, the Arkansas River meandered freely through its flood plain, with bends of alternating curvature that generally migrated downstream, elongating their loops to where natural cutoffs occurred. High and low flows generally took different paths around the bends, with low flows following a sinuous path in the main channel and higher flows cutting across the point bars. From Robert S. Kerr Dam, Oklahoma, to the river mouth, radius of curvature ranged from 1,000 ft to about 36,000 ft;

50 percent of the concave banks had radii of 6,000 ft or less, and 80 percent had radii of 10,000 ft or less. The meander pattern of an alluvial stream has been associated with a "dominant" discharge. On the Arkansas River that flow was considered to be the discharge at approximately bankfull stage, ranging from 150,000 cfs at Fort Smith to about 275,000 cfs below Pine Bluff, discharges equaled or exceeded about five percent of the time and with approximately an annual recurrence interval under natural conditions. The meander pattern is also influenced by slope of the stream, sediment load, and relative erodibility of bed and banks.

For design of the Arkansas River project, studies were made of the shape of naturally stable bends and crossings at various points on the river to determine what design widths the stream could be expected to maintain without developing center bars and split channels. Average values of cross-sectional area and carrying capacity ($Ad^{2/3}$) were determined for a range of flows in the stable sections and used as a guide for design of rectified sections at other nearby locations. Studies were also made of average maximum depth of channel cross section in bends as a function of radius of curvature of the bend and of depths in crossings as a function of width. For example, in the Little Rock to Pine Bluff reach, maximum depth in bends decreased from about 20 to 10 ft below the construction reference plane* as radius increased from 5,000 to 35,000 ft. Depths in crossings decreased from 10 to 5 ft below the construction reference plane as width increased from about 1,100 to 1,600 ft, as shown on Figure 1-3.

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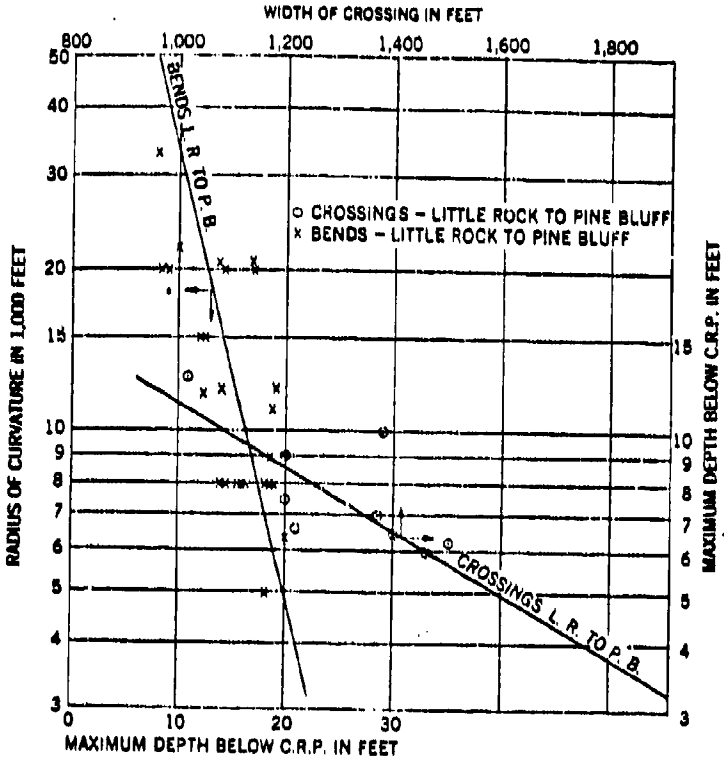
* The construction reference plane (CRP) is the vertical control line on the Arkansas River, a sloping plane corresponding approximately to the mean low-water profile for a flow of 10,000 cfs for preproject conditions and adjusted for river shortening under project conditions.



Arkansas River
Maximum Depths in Crossings
and Stable Bends
Little Rock to Pine Bluff

Figure I-3

The Arkansas River was stabilized in a single channel along an alignment consisting of a series of easy bends of alternating curvature, usually connected by straight crossing reaches about two to four times as long as the width. The layout followed the natural configuration of the river as much as practicable so as to minimize disturbance of the river regime and minimize cost. Where bends were very sharp, cutoffs were constructed because initial and maintenance costs would be lower than for stabilization of the natural bends and navigation conditions would be improved. Development of long, relatively straight reaches was avoided where possible since there is a tendency for flows to shift from side to side in such reaches, making maintenance expensive and creating difficulties for navigation.

Since realignment involved local scour and deposition along much of the river over a period of several years (depending on the extent of realignment in a given reach and future flow magnitude), that work was programmed for early construction so the high sediment load in the river prior to closure of the Oklahoma storage reservoirs could be utilized in reshaping the channel.

Cutoffs were constructed by excavating a pilot channel of relatively small cross section which was later enlarged by the river itself. The pilot channel cross section was designed to be self-scouring, that is, so the sediment transporting capacity of the cut was greater than required to transport sediment entering the cut and the pilot channel would degrade and enlarge. For initial cutoffs, low flows that would have tended to deposit material in the excavated cuts were prevented from entering the pilot

channels immediately after excavation by leaving an earth plug near the upstream end that was designed to be overtopped and washed out at a flow of about 100,000 cfs. For the later cutoffs, when time for development prior to impoundment of the navigation pools became short, the excavated cuts were opened to the river immediately after construction so that all flows would contribute to enlargement of the cuts.

If a sufficiently high bed-shear ratio to ensure enlargement of the pilot channel could not be obtained with an economical excavated cross section, the slope through the cut was increased by raising the water surface elevation at the head of the cut by construction of a control structure across the old channel just downstream from the entrance to the cutoff. In most cases, the structure initially extended across only a part of the old channel. If it traversed the entire cross section, it was designed with a long spillway section at a low elevation so that a substantial portion of the flow continued through the old bend for a time, depositing sediment in the old bendway. Such structures are required ultimately to control alignment of the channel, but by constructing them shortly after a cutoff was opened, they also promoted more rapid development of the cutoff than would naturally occur. A revetment was constructed along the rectified channel control line on the concave bank of the cutoff to limit and control enlargement of the channel.

Major realignment of the river in Arkansas involved construction of 15 cutoffs (11 downstream of Dardanelle Dam). The river between Dardanelle Dam and Dam 2 was shortened by 35 miles, or 16 percent. Table 1-4 summarizes information on cutoffs and pilot channels on the Arkansas River

Table 1-4

Pertinent Data, Arkansas River Pilot Channels in Arkansas

Pilot Channel (1)	1940 River Mile (2)	Date Opened to River (3)	L_{pc}, in feet (4)	L_r (5)	Shortening, in miles (6)	Rectified Radius, in feet (7)	Excavated Bottom Width, in feet (8)	Bottom Grade[c] (9)	Excavation, in million cu yd (10)	Soils Characteristics (11)	Cleared Width, in feet (12)
Shoofly Bend	358.5	4 Sept '62	9,000	1.15	0.23	8,000	200	-5	1.27	Bottom: poorly-graded sands Sides: silts, silty sands	....
Trustee Bend	339	May '54	8,150	2.07	1.65	13,500	50	-1	0.35	Bottom: sands and silty sands Sides: clays and silts	400
Arbuckle Island	333	6 Sept '62	10,300	1.9	1.76	10,600 8,200	50	-7	1.44	Bottom: poorly-graded, silty sands Sides: silts, sands fine to grav.	400
McLean Bottom Additional excav.	296	Mar '55 Sept '56	7,800	2.06	1.58	12,700	50 30	-1 -7 to -10	0.59 0.21	Bottom: poorly-graded sands Sides: silts and sandy silts	400
Point Bar Additional excav.	250.5	May '51 Jan '57	5,500	1.72	0.76	7,500	20	0 -4	d 0.24	Bottom and sides: largely poorly-graded sands	400
Holla Bend Additional excav.	249.6	May '54 Nov '54	14,000	3.38	6.30	25,000	20 33	+1 to +2 -5.5 to -7.5	d 0.15	Bottom: sands Sides: fat and lean clays, silts	400
Ellis Island	219.5	25 May '62	6,450	1.05	0.57	17,750	130	-10	1.76	Bottom and sides: poorly-graded and gravelly sands	....
Morrilton	214.5	13 May '50	5,000	6.0	5.20	17,000	40	-4 to -6	0.33	Bottom: sands Sides: silts, sands, some clay	1,000
Fourche Place	158.5	5 Feb '62	8,200	1.86	1.35	8,400	130	-7	1.81	Bottom: poorly-graded sands Sides: sands, silts, clay lenses	400
Willow Bar	156.4	3 Aug '62	9,800	1.08	0.15	19,600	130	-10	1.79	Bottom and sides: poorly-graded sands	....
Case Bar	145.5	28 Nov '62	19,500	1.75	2.80	8,000 11,400	100	-10	2.49	Bottom: poorly-graded sands Sides: silts, sands, clay lenses	400
Brodie Bend	137.0	5 Apr '57	13,300	3.20	5.55	8,000	50	0 to -3.5	0.90	Bottom: poorly-graded sands Sides: silts, sands, clays	400
Hensley Bar	120.0	Nov '51	11,100	1.46	0.97	20,800	100	-6	1.76	Bottom: sands Sides: silts, sands, clays	400
Boyd Point	101.0[a]	Sept '62	16,000	2.9	5.75	8,000	100	-8	....	....	....
Little Bayou Meto	68.0[a]	30 Aug '62	10,500	3.83	5.62	5,800	100	-8	1.10	Bottom: sands and silts Sides: silts, sands, clays	650

[a] 1943 river mile (1940 mile 115.7 equals 1943 mile 101.7)

[c] In feet with respect to CRP (construction reference plane, corresponding approximately to the water surface profile for a flow of about 10,000 cfs

[d] Point Bar and Holla Bend original excavation 532,000 cu yd

in Arkansas. It will be noted that all cutoffs were opened by the end of 1962, or prior to closure of Dardanelle Dam and the low-lift navigation dams downstream, permitting much of the material eroded as the pilot channels enlarged to move through the system prior to impoundment of the pools.

In layout of the rectification work, the bends were designed with smooth transitions to tangents in the crossings, and the revetments were laid out free of irregularities and false points to avoid separation of flow and localized scour from eddy action. The original layout required rather extensive control structures on the concave bank in bends, minor work on convex bars, and work on both banks through crossings.

In general, the rectified alignment was achieved by spur dikes extending out from existing banks to the rectified channel control line. Where the control line was along concave banks, the banks were stabilized by construction of revetments. The original trace width design ranged from 1,000 to 1,500 feet in crossings. In bends, the stream had more flexibility in developing its own optimum width, and the spur structures on the convex banks were designed to be extended out into the channel in the future as required.

The low-head navigation dams were spaced as far apart as feasible to minimize costs, and additional contraction (additional narrowing) was provided at the heads of pools to produce and maintain the design navigation depth.

Operating experience later indicated the need for additional contraction in some reaches to minimize maintenance dredging and provide more reliable navigation depths, as discussed by pool in Chapter 2.

Experience had shown that stabilization of an alluvial stream such as the Arkansas must be achieved by systems of protective works extending from one fixed point on the stream to another because isolated structures become ineffective in a short time as changes in flow direction destroy the structures or migration of the river bypasses the structures and shifts the attack to a new location requiring work in another area. The location and composition of the systems of structures used on the Arkansas were determined largely on the basis of experience, judgment, and the application of rules deduced from years of river observation.

In general, protection was provided to the stage corresponding to about half-bankfull discharge (which is a flow equaled or exceeded about 10 percent of the time). Bank paving was carried generally either to about mid-bank height (16 ft above the construction reference plane) or to the top of bank. Top of piling was usually set at from 13 to 17 ft above the construction reference plane, and as piling deteriorated over time, additional stone was added to pile structures to an elevation of 10 feet above CRP. The specific criteria used at a particular site depended on local conditions.

Four types of revetment were used for bank stabilization on the Arkansas River, with the type used at a specific location depending on purpose of the structure, depth to bed rock below the stream bed (which sometimes precluded driving piling), and the relative cost of stone and pile structures. Where existing concave banks were to be protected from erosion, either standard trenchfill revetment or standard revetment with mattress was used, depending on relative cost. On low banks, low bars, and where the

rectified channel control line crossed the existing channel, either pile revetment or stone-fill revetment was used, depending on the elevation of top of rock and relative cost. These two types of revetment were designed to induce deposition of sediment to create new bank lines or build up existing banks and to encourage willow growth on the accretions. Where the rectified control line was a considerable distance from the existing bank, baffle dikes were used landward of the revetment to provide additional screening effect. Design of revetments was based on standards described in Project Design Memorandum No. 8, Bank Stabilization and Channel Rectification, Arkansas River, 1960.

Both stone-fill and pile dikes were used, with the type used in a specific location depending on depth of bed rock below the stream bed and relative costs. Either type was satisfactory to control the river, train it to the rectified alignment, promote the development of new banks, close off secondary channels, and serve as baffles landward of pile or stone-fill revetments. Steel jetties were used to a very limited extent to expedite accretion and growth of willows in chutes, on the upstream ends of bars, and on low overbanks, primarily upstream from Lock and Dam 13.

Dike systems on concave banks usually consisted of a leadoff structure at the upstream end (approximately parallel to the flow and far enough upstream so that flanking did not threaten integrity of the system) and a series of spur dikes extending out from the natural bank to the rectified channel control line. The spur dikes were angled downstream about 15 degrees from normal to the current to direct flow away from the dike root and back to the channel. Where spur dikes were short and the degree of

attack was not severe, as in a relatively straight reach or flat bend, the maximum spacing of dikes was such that flow around the ends of the dikes did not attack the bank between them. Where the dikes were longer and the degree of attack more severe, as in bends of moderate to sharp radius, the spacing was usually from one to 2.5 times the dike length, with the maximum spacing such that not more than the outer 20 to 25 percent of any dike was subject to direct attack by the current. Longer dikes were set relatively closer together. The closer spacing resulted in lower maintenance costs and minimized the scalloped bankline pattern that accretions form behind dikes, reduced turbulence, and improved flow conditions for navigation. Where an existing concave bank was faired out riverward a considerable distance to the rectified control line, the dikes sometimes were extended in stages to allow time for the new channel to develop past the ends of the dikes at each stage. When this was done, dikes were spaced to meet the above criteria at the intermediate construction stages (as well as for the ultimate alignment) because several years usually elapsed between stages, and good control was required throughout the entire realignment period. Dikes were used on the convex side of bends to promote accretion on convex bars, close off chutes and swales, and prevent development of new chutes. In general, they had a much greater spacing than dikes on the concave side and were used primarily in the upper third of a bend. Design standards for dikes are also discussed in Project Design Memorandum No. 8 referenced above.

Operation of Navigation Dams - Spillway crests for the low-lift navigation dams were set at about the estimated future stable bed level. On rising stages, planned operation provided for opening the spillway gates as

rapidly as possible, consistent with maintaining navigable depths, so that essentially open-river conditions would prevail at medium-to-high discharges. Thus, the river at higher flows (with spillway gates fully open) was projected to retain approximately its natural sediment transport capacity, with no overall tendency for deposition in the navigation channel in the lower river. On falling stages, spillway gates were to be closed to maintain navigable depths, detention time through the pools would be increased, and an increased percentage of the sediment load would be deposited. The minimum flow at which open-river conditions prevail are discussed further in Chapters 2 and 6.

Because low flows carried a low sediment load, deposition in the pools of the low-lift navigation dams was expected to be minor, and some of the material deposited in the pools during low flows was expected to be entrained and moved downstream on the next rise during open-river conditions. Thus, longer periods between flood events were expected to increase maintenance dredging. It was estimated that the silt-clay load passing Dardanelle Dam would be too fine to be deposited in the downstream low-lift pools, as noted earlier.

Spillway operating criteria generally call for all gates to be open uniformly when passing flows in excess of those required for lockage. A hinged pool operation has been used at some dams to increase slopes and velocities in the lower portion of a pool so that any deposited material will be scoured out; flow conditions in the upper portion of the pool are not affected. The pool is drawn down from 2 to 5 ft below normal navigation pool level at the dam by opening the spillway gates when river discharge

is increasing. The amount of drawdown is limited so that the water surface profile through the pool is at about project design depth through controlling shallow obstructive reaches and velocities are not so high as to restrict navigation in the lower reach of the pool. An alternative operating procedure to maintain project depth in shoaled areas is to raise the pool above normal elevation temporarily until the shoaled area can be dredged. The permissible increase in pool level is controlled by the extent of flooding of adjacent lands and freeboard (1 to 2 ft) on the spillway gates.

CHAPTER 2 - LOW-LIFT NAVIGATION POOLS BELOW DARDANELLE DAM

2-1 GENERAL

Pools 9 through 2 downstream from Dardanelle Dam have very different characteristics at normal pool with regard to:

- Storage, ranging from 110,000 ac ft at Pool 2 to 32,000 ac ft at Pool 8.
- Pool length, ranging from 33.2 mi at Pool 2 to 15.8 mi at Pool 3.
- Surface area, ranging from 10,500 ac at Pool 2 to 3,700 ac at Pool 3.
- Average pool depth, (storage divided by area at normal pool level), ranging from 12.4 ft at Pools 3 and 4 to 7.6 ft at Pool 8.
- Elevation of the construction reference plane (CRP) with respect to pool elevation, ranging from +8 ft to -8.5 ft at the head of Pools 9 and 3, respectively, and -26 ft to -13.5 ft just upstream from Dams 2 and 8, respectively.
- Minimum discharge at which all spillway gates are fully open, ranging from 80,000 cfs at Dam 8 to 280,000 cfs at Dam 2 .

All these factors affect the efficiency of stabilization and rectification work in providing a stable navigable channel of adequate depth. Pertinent characteristics of the pools are summarized in Table 2-1.

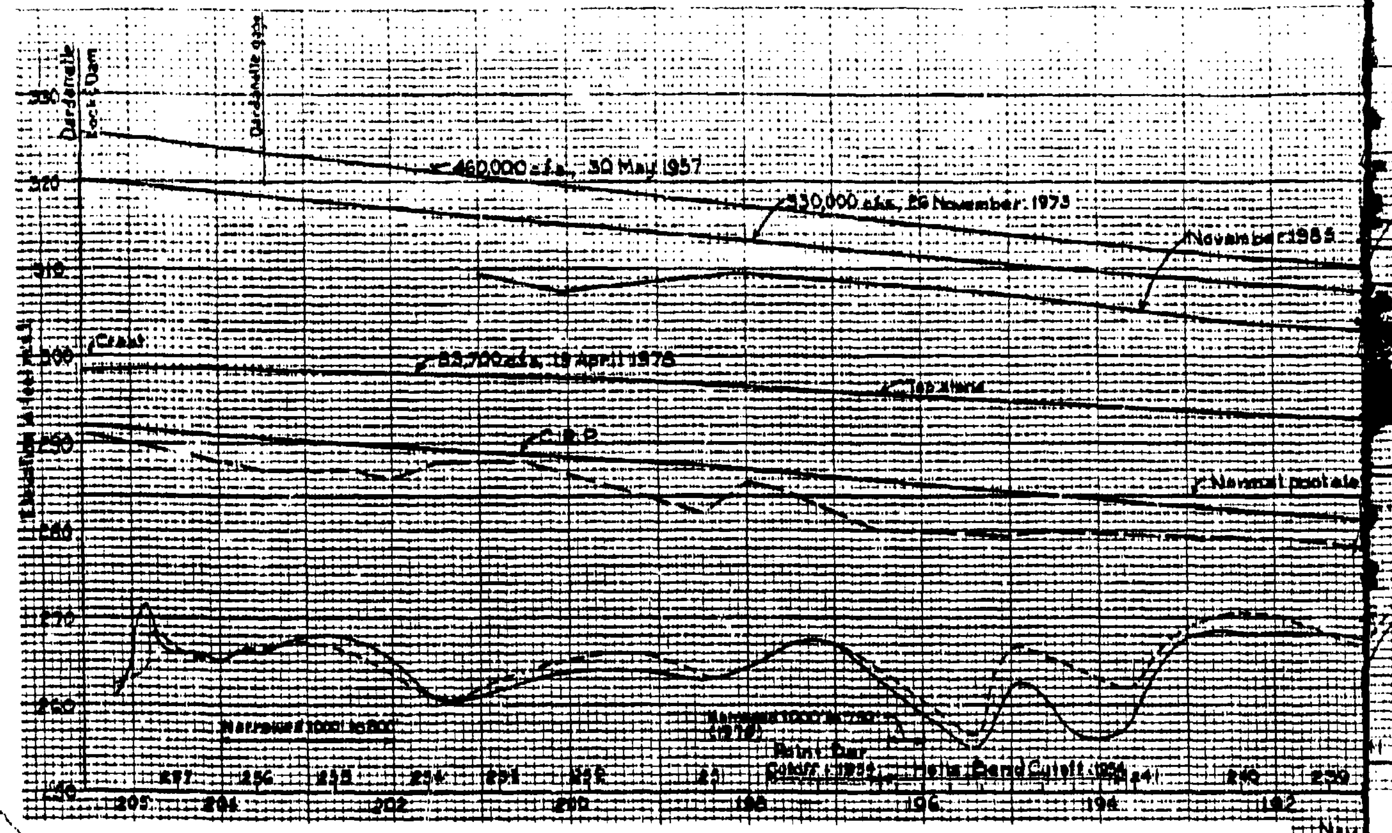
The layout and profile for Pools 9 through 2 are shown on Figures 2-1 through 2-8, based on available information as listed in Table 2-2. The figures show initial stabilization and rectification work and structures added after the reach became operational in December 1968 to further narrow the channel to reduce maintenance dredging and provide more reliable

Table 2-1

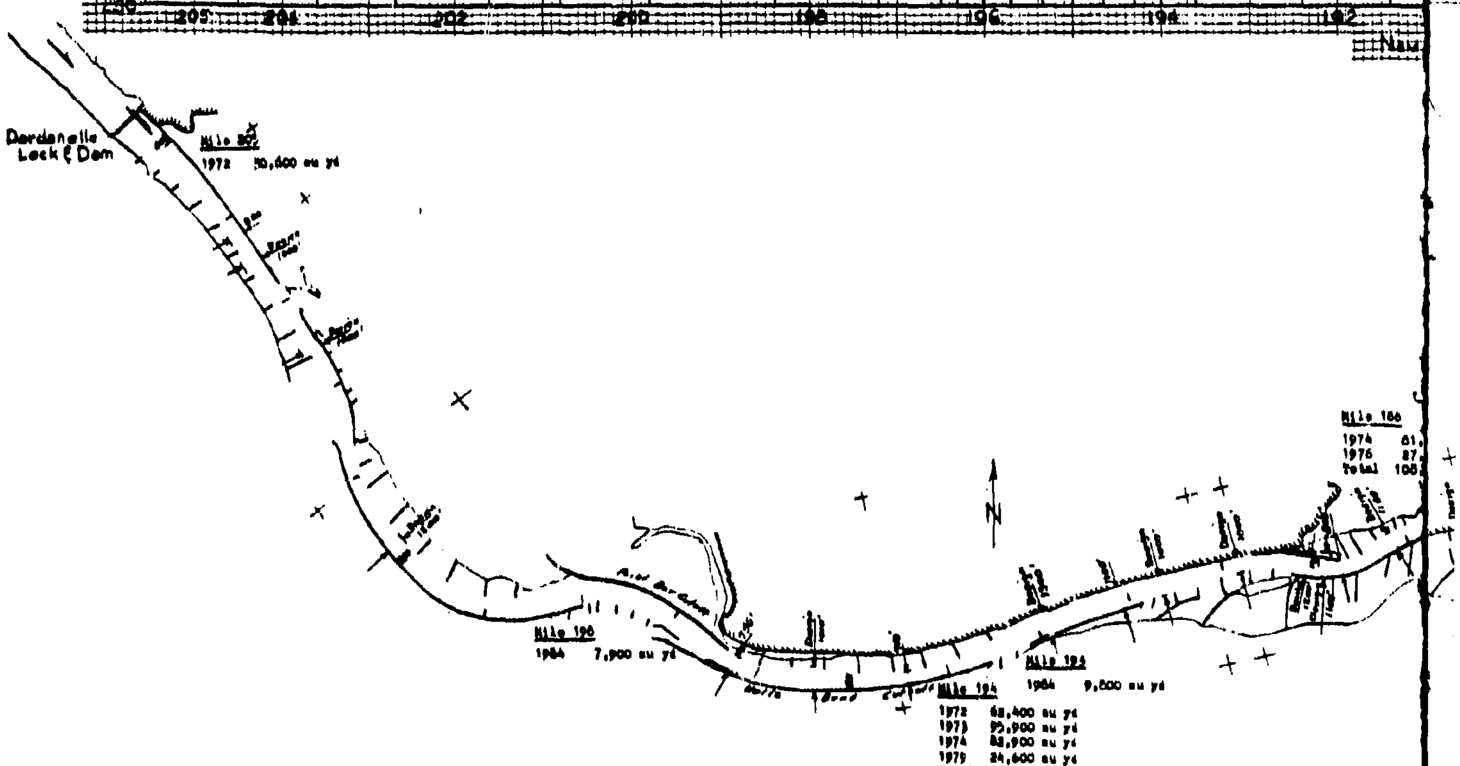
Pool Characteristics at Normal Pool Elevation

Pool	Normal Pool (m.s.l.)	Storage (ac ft)	Length (Nav mi)	Area (ac)	Avg Depth(1) (ft)	CRP Elevation (2) Head of Pool	CRP Elevation (2) Dam	Approximate Open-River Discharge (1,000 cfs)	Approximate Open-River Discharge Return Interval (years)(5)
13	392	59,000	27.0	6,820	8.7			125-127	
Ozark	372	148,000	36.0	10,600	14.0			510	
Dardanelle	338	430,000	51.3	34,300	12.5			1,500	
9(3)	284	56,000	28.6	4,900	11.4	+ 8.0	-17.5	137	1.3
	287	69,000	28.6	5,650	12.2				
8	265	32,000	21.0	4,200	7.6	+ 1.5	-13.5	80	1.1
7	249	86,000	30.5	9,750	8.8	+ 3.5	-21.5	190-200	2.4
6	231	50,000	17.3	4,700	10.6	- 3.5	-17.5	155	1.6
5	213	61,000	21.8	6,700	9.1	+ 0.5	-20.5	145-150	1.4
4	196	70,000	20.3	5,650	12.4	- 4.0	-22.5	145-150	1.4
3	182	46,000	15.8	3,700	12.4	- 8.5	-18.2	125	1.3
2	162	110,000	33.2	10,500	10.5	+ 1.5	-26.0	280(4)	7.4

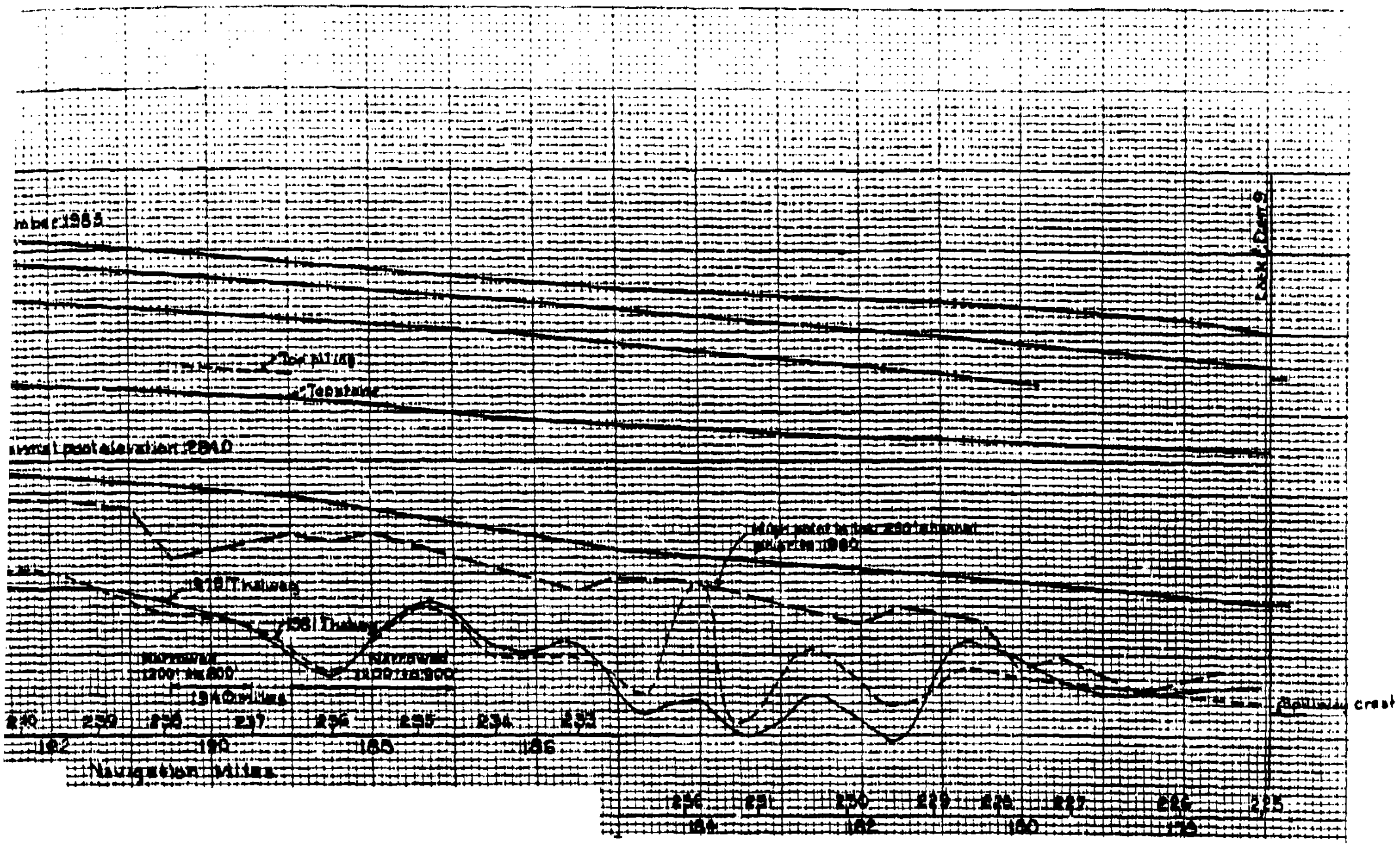
(1) Storage divided by surface area at normal pool elevation.
(2) CRP elevation with respect to normal pool elevation.
(3) Pool fluctuates from elevation 284 to 287 during low-flow periods to modify surges from Dardanelle power releases.
(4) Low Mississippi.
(5) Based on data for 22-year period 1964-1985 after Dardanelle Dam became operational.

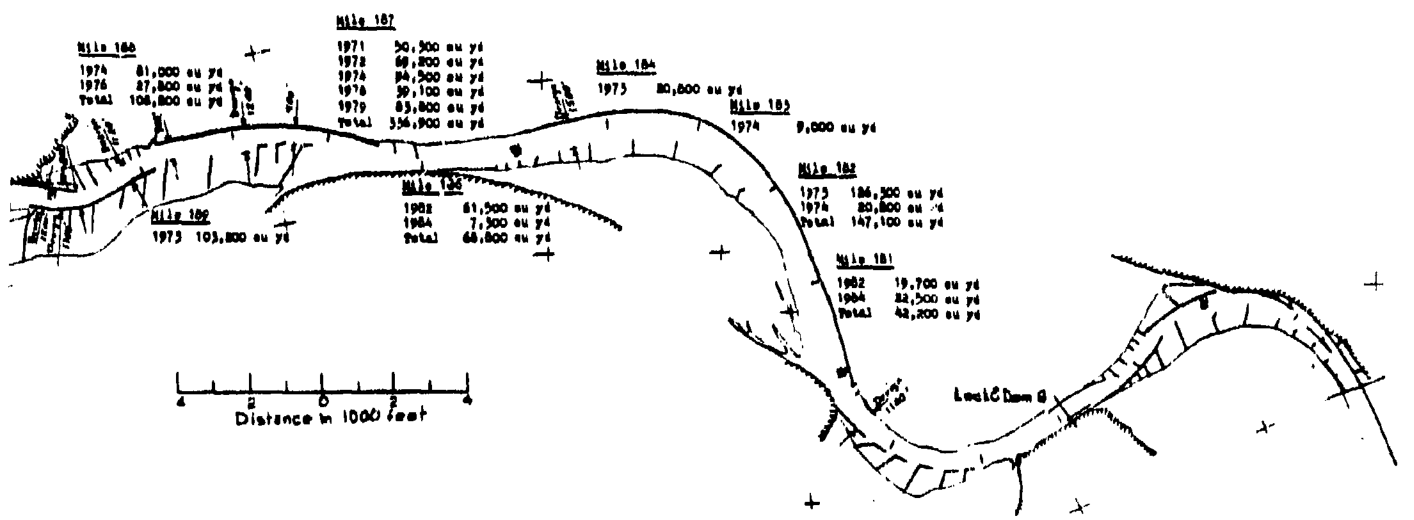


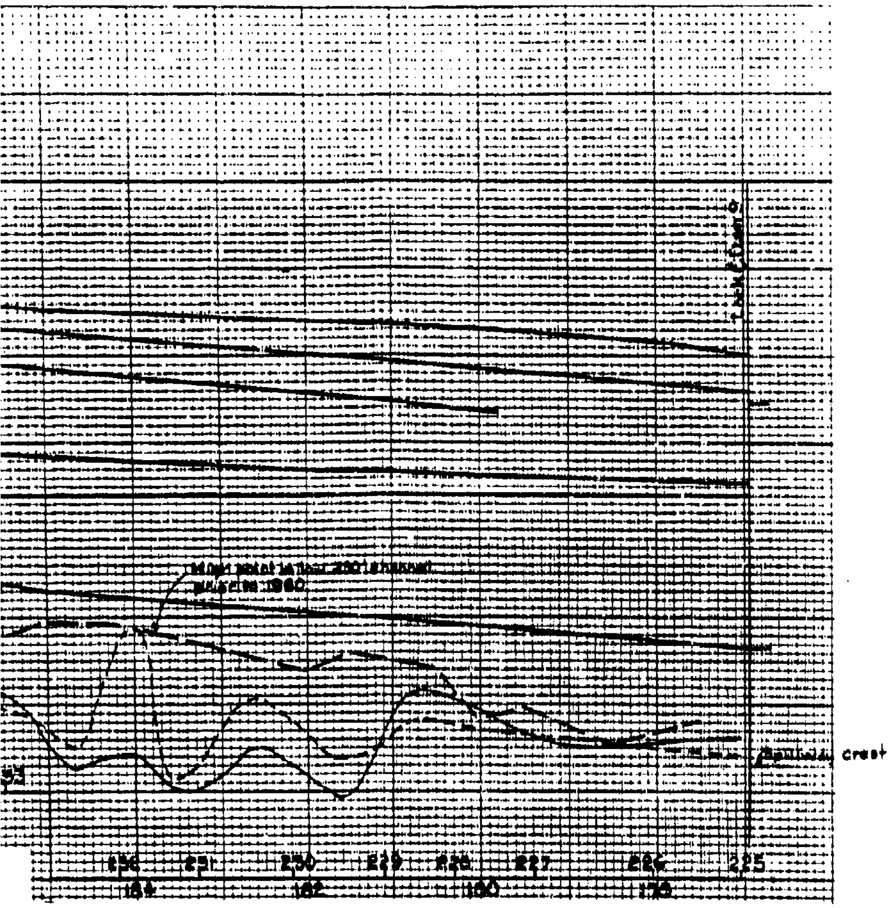
Dardanelle Lock & Dam
460,000 c.f.s., 30 May 1957
November 1985
Crest
C.R.P.
Normal pool

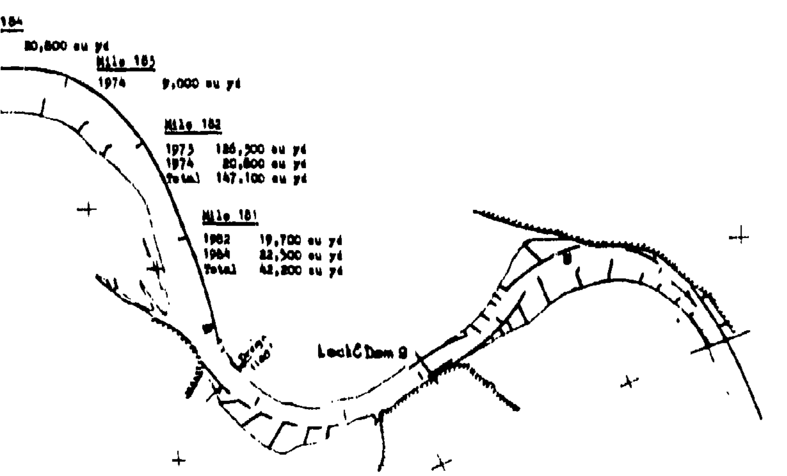


Dardanelle Lock & Dam
1972 50,000 cu yd
Mile 196
1984 7,900 cu yd
1984 9,800 cu yd


Top piling
Top stone
1918 Thalweg
1961 Thalweg
Spillway crest
Navigation Miles
240
239
238
237
236
235
234
233
192
190
188
186
236
231
230
229
228
227
226
225
184
182
180
178


Mile 187
1971 50,500 cu yd
1974 94,500 cu yd
1979 85,800 cu yd
Total 336,900 cu yd
Mile 184
1975 80,600 cu yd
Mile 183
1974 9,000 cu yd
Mile 182
1975 126,300 cu yd
Total 147,100 cu yd
Mile 181
1982 19,700 cu yd
Total 42,200 cu yd
Mile 189
1973 103,800 cu yd
Distance in 1000 feet
Lock & Dam 9





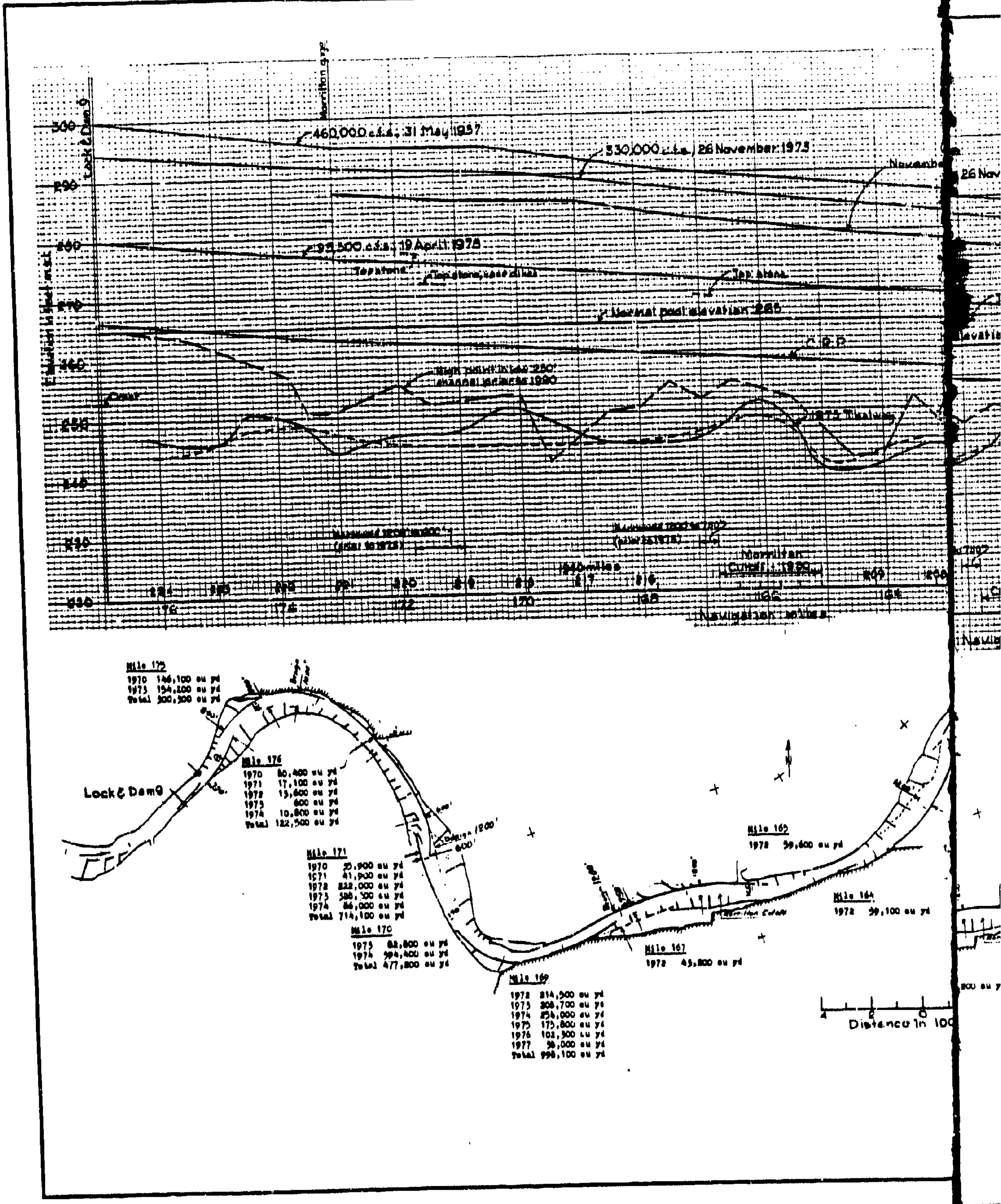
Arkansas River, Arkansas

Layout and Profiles

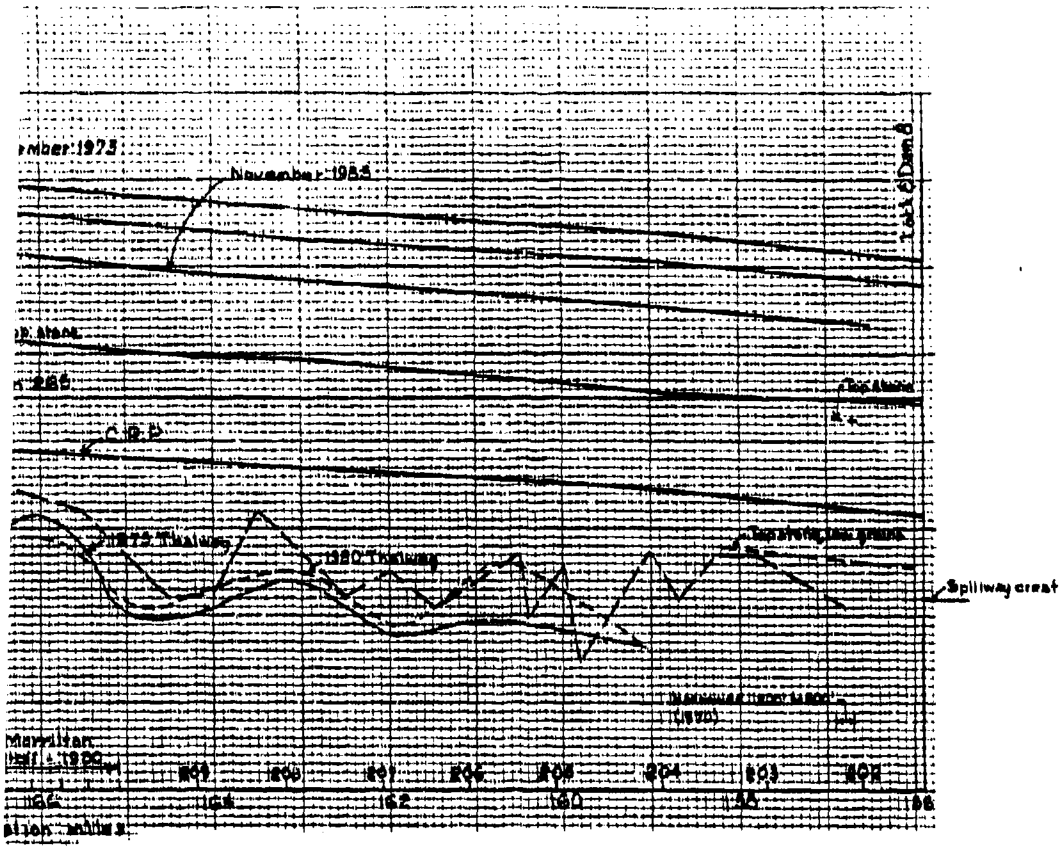
Pool 9

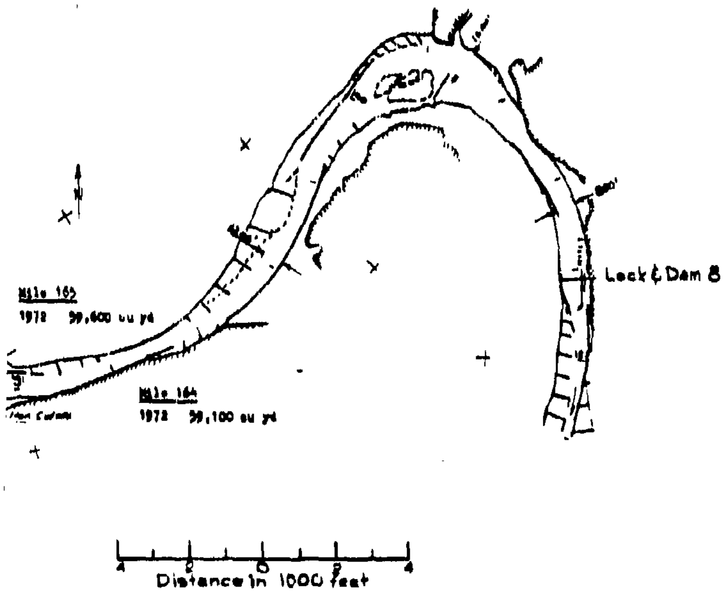
N. Mile 205.5 to N. Mile 176.9

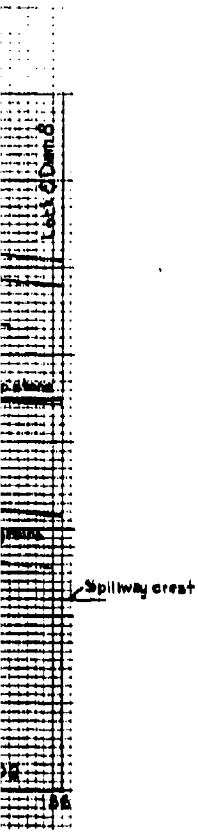
Figure 2-1



Lock & Dam 9
Morrilton gage
460,000 c.f.s. 31 May 1957
330,000 c.f.s. 26 November 1973
195,500 c.f.s. 19 April 1978
Top stone
Top stone
Normal pool elevation 265
C.R.P.
High point in top 250' channel prism in 1980
1975 Thalweg
Morrilton Cutoff 1990
Navigation miles
300
290
280
270
260
250
240
230
176
174
172
170
168
166
164
Mile 175
1970 146,100 cu yd
1973 154,200 cu yd
Total 300,300 cu yd
Mile 176
1970 80,400 cu yd
1971 17,100 cu yd
1972 13,600 cu yd
1973 600 cu yd
1974 10,800 cu yd
Total 122,500 cu yd
Mile 171
1970 55,900 cu yd
1971 41,900 cu yd
1972 822,000 cu yd
1973 586,300 cu yd
1974 86,000 cu yd
Total 714,100 cu yd
Mile 170
1973 82,800 cu yd
1974 394,400 cu yd
Total 477,800 cu yd
Mile 169
1972 814,500 cu yd
1973 808,700 cu yd
1974 256,000 cu yd
1975 175,800 cu yd
1976 102,300 cu yd
1977 38,000 cu yd
Total 998,100 cu yd
Mile 167
1972 43,800 cu yd
Mile 165
1972 59,600 cu yd
Mile 164
1972 59,100 cu yd
4
2
0
Distance in 100


November 1955
1920 Thalweg
Spillway crest
Lock & Dam 8
164
162
160


Lock & Dam 8
Mile 165
1972 59,600 cu yd
Mile 164
1972 59,100 cu yd
Distance in 1000 feet



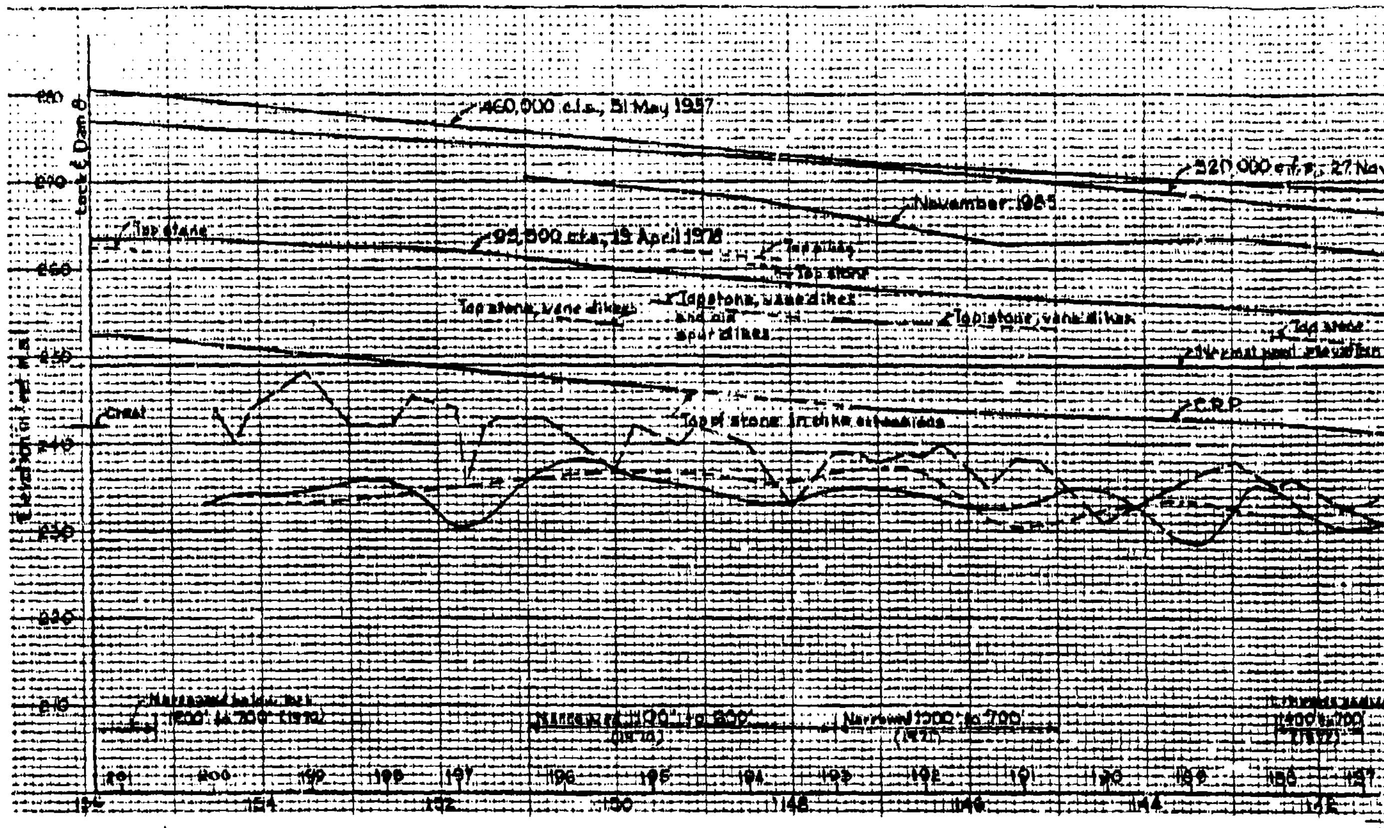
Arkansas River, Arkansas

Layout and Profiles

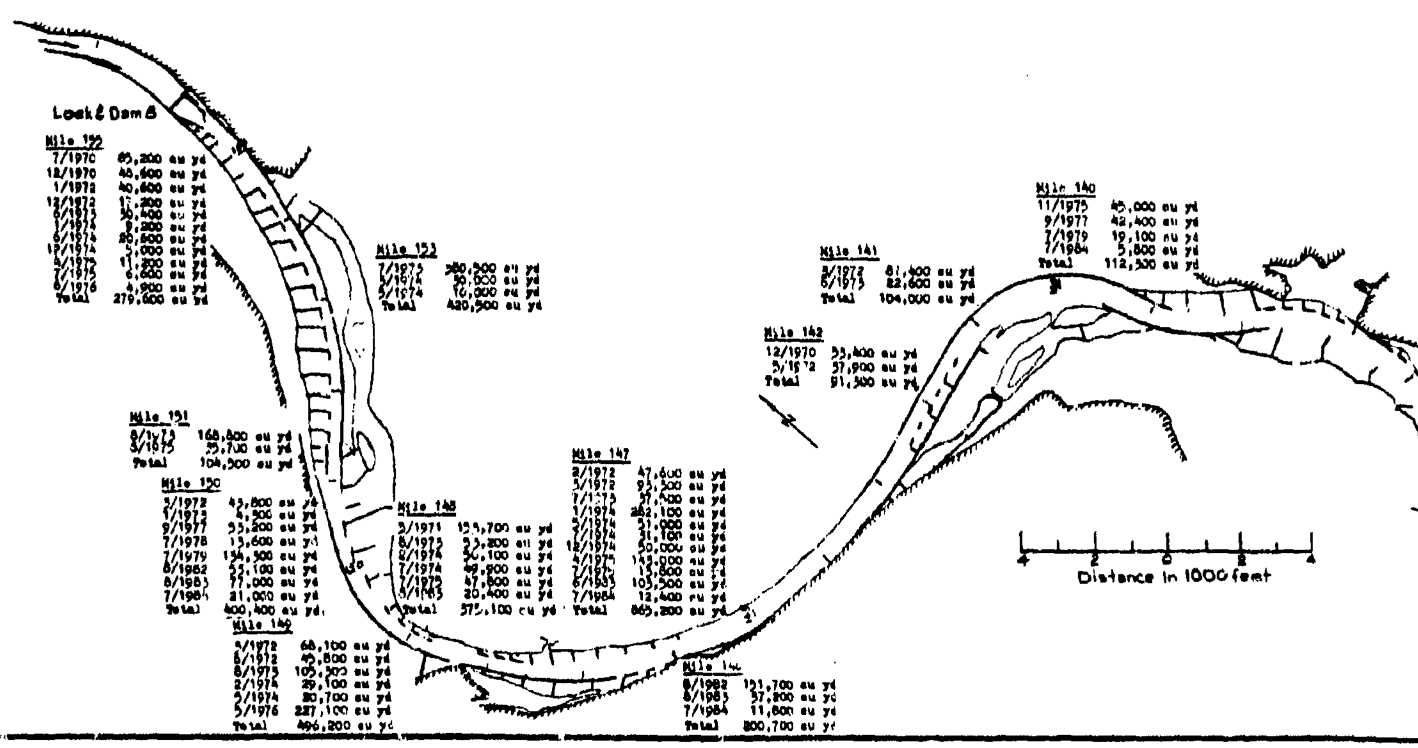
Pool 8

N. Mile 176.9 to N. Mile 155.8

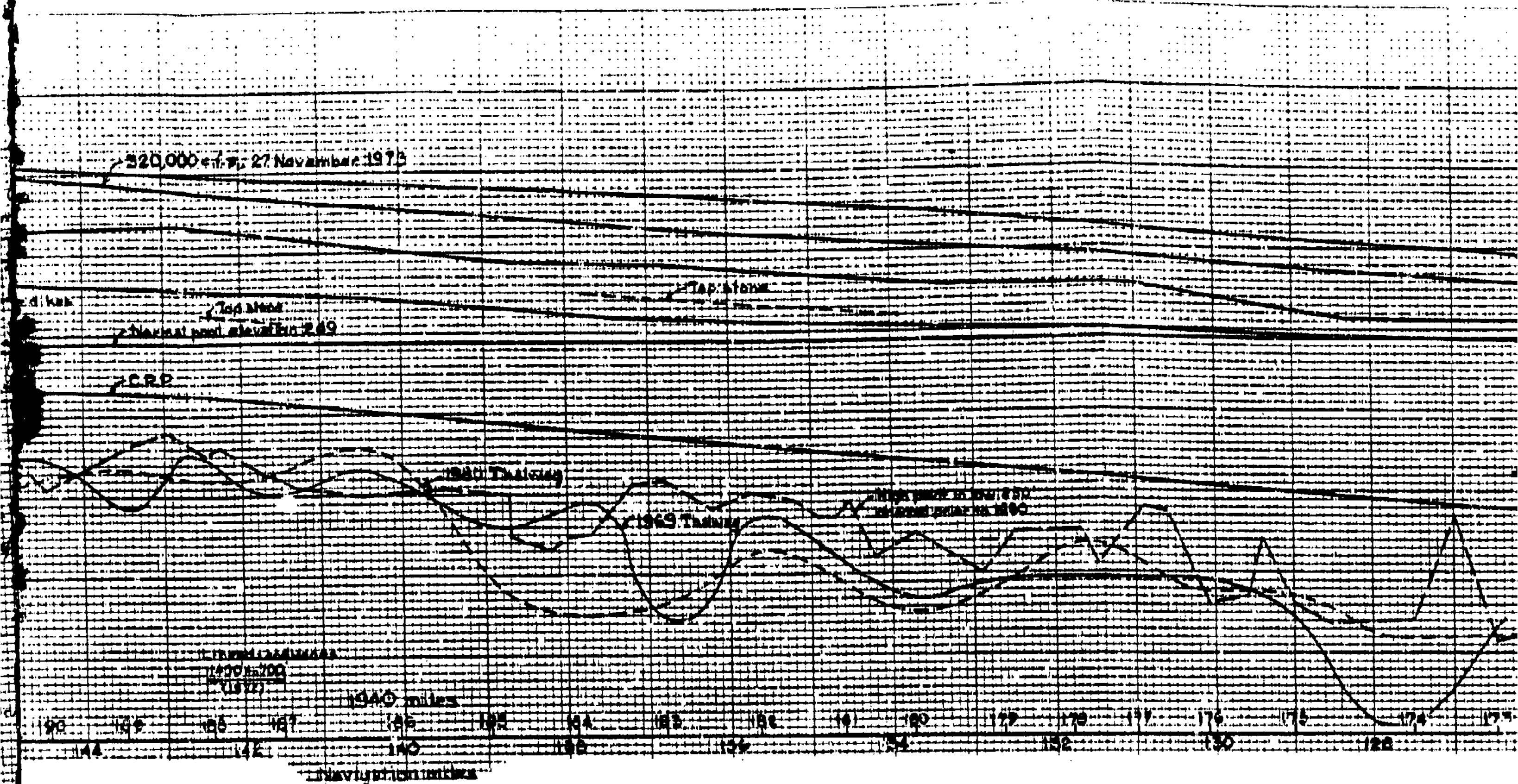
Figure 2-2



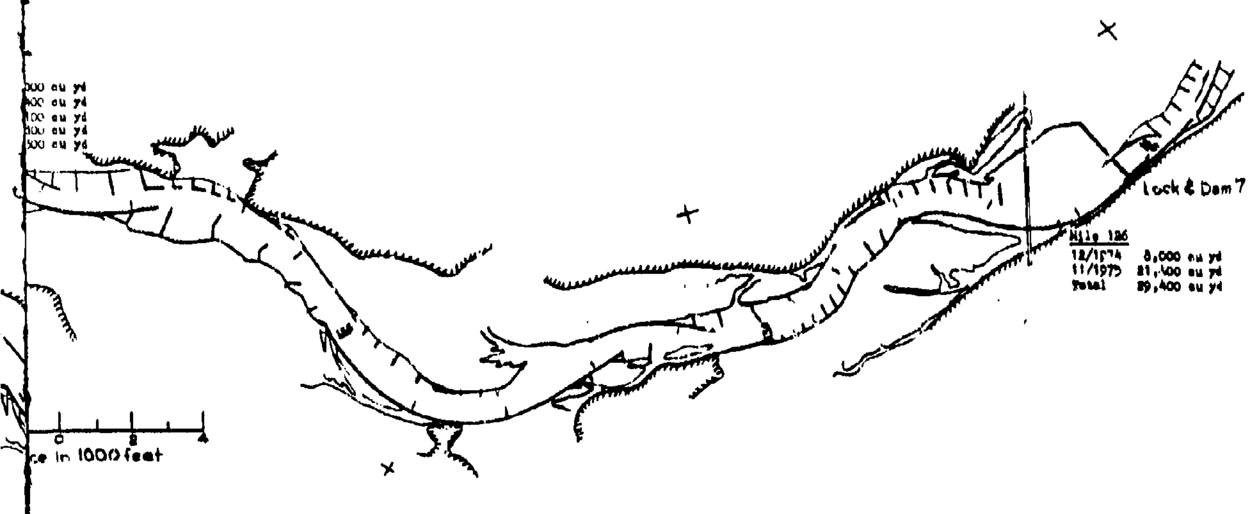
Lock & Dam 8
460,000 c.f.s., 31 May 1957
November 1985
95,500 c.f.s., 19 April 1978
Top stone
Top stone, vane dikes
C.R.P.
Crest
Elevation in feet m.s.l.



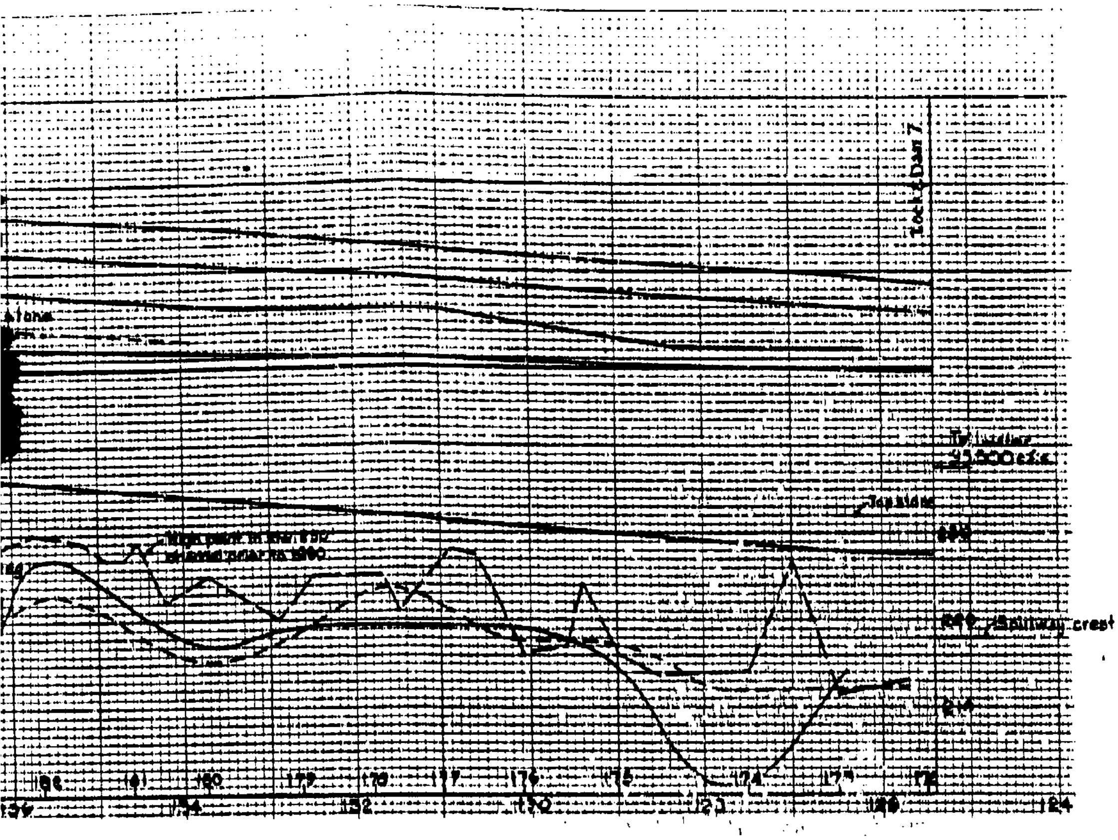
Lock & Dam 8
Mile 152
7/1970 65,200 cu yd
12/1970 46,600 cu yd
1/1972 40,600 cu yd
Total 279,600 cu yd
Mile 153
7/1973 380,500 cu yd
Total 420,500 cu yd
Mile 151
8/1973 168,800 cu yd
Total 104,500 cu yd
Mile 150
3/1972 43,800 cu yd
9/1977 55,200 cu yd
7/1978 13,600 cu yd
7/1979 134,500 cu yd
7/1984 21,000 cu yd
Total 400,400 cu yd
Mile 148
5/1971 155,700 cu yd
8/1973 53,200 cu yd
7/1974 49,900 cu yd
7/1975 47,800 cu yd
Total 375,100 cu yd
Mile 149
8/1973 105,300 cu yd
2/1974 29,100 cu yd
5/1974 20,700 cu yd
5/1976 227,100 cu yd
Total 496,200 cu yd
Mile 147
2/1972 47,600 cu yd
5/1972 93,300 cu yd
1/1974 282,100 cu yd
12/1974 50,000 cu yd
4/1975 145,000 cu yd
6/1983 103,500 cu yd
7/1984 12,400 cu yd
Total 865,200 cu yd
Mile 144
8/1982 151,700 cu yd
8/1983 37,200 cu yd
7/1984 11,800 cu yd
Total 800,700 cu yd
Mile 142
12/1970 53,400 cu yd
Total 91,300 cu yd
Mile 141
3/1972 81,400 cu yd
6/1973 22,600 cu yd
Total 104,000 cu yd
Mile 140
11/1975 45,000 cu yd
9/1977 42,400 cu yd
7/1979 19,100 cu yd
7/1984 5,800 cu yd
Total 112,500 cu yd
Distance in 1000 feet

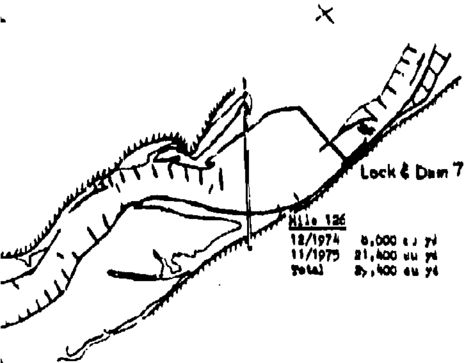


320,000 c.f.s., 27 November 1973
Top stone
Top stone
Normal pool elevation 249
CRP
1960 Thalweg
1969 Thalweg
1940 miles
Navigation miles



Lock & Dam 7
Mile 126
12/1974 8,000 cu yd
11/1975 21,400 cu yd
Total 29,400 cu yd
0 2 4
ce in 1000 feet



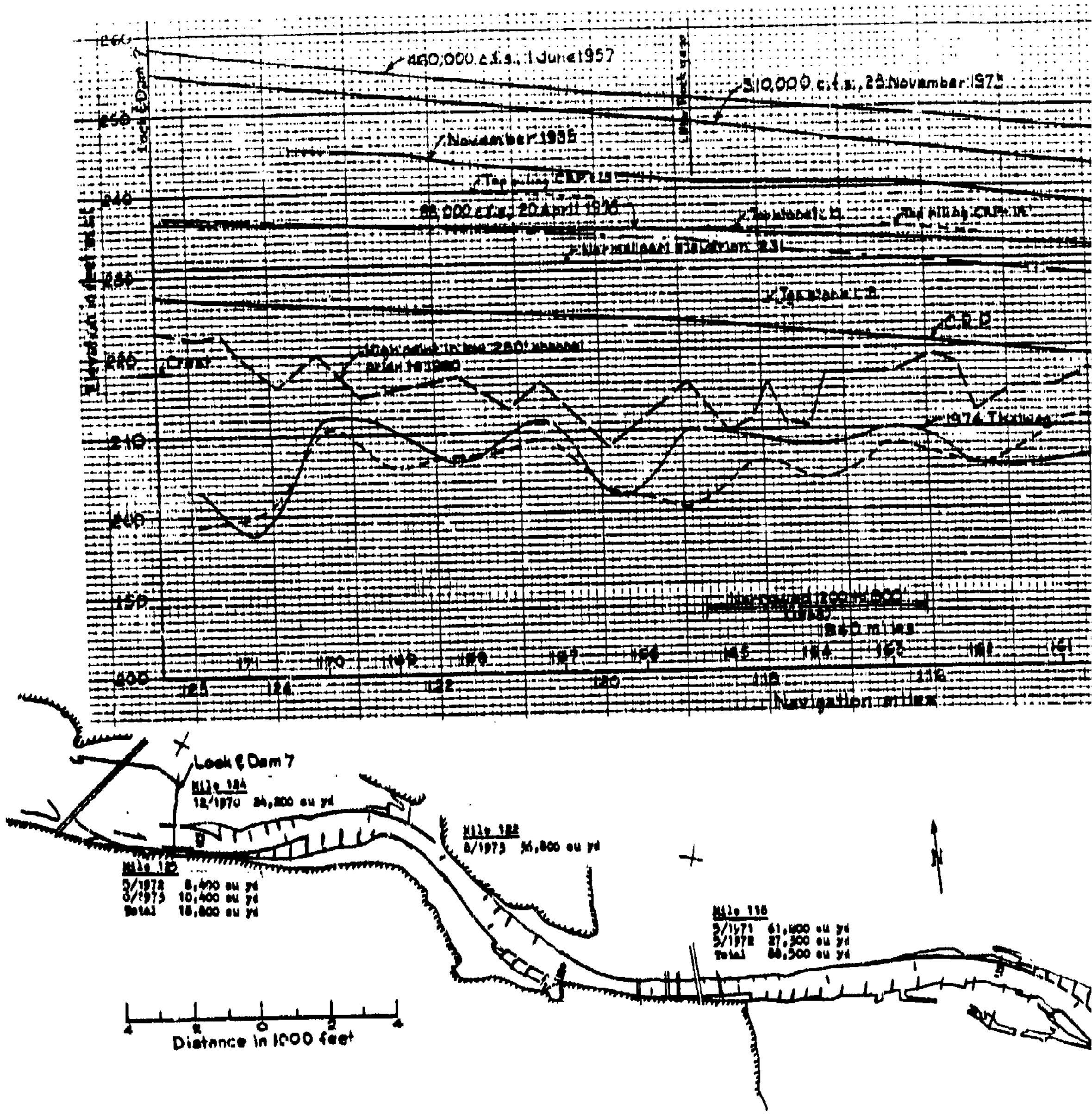


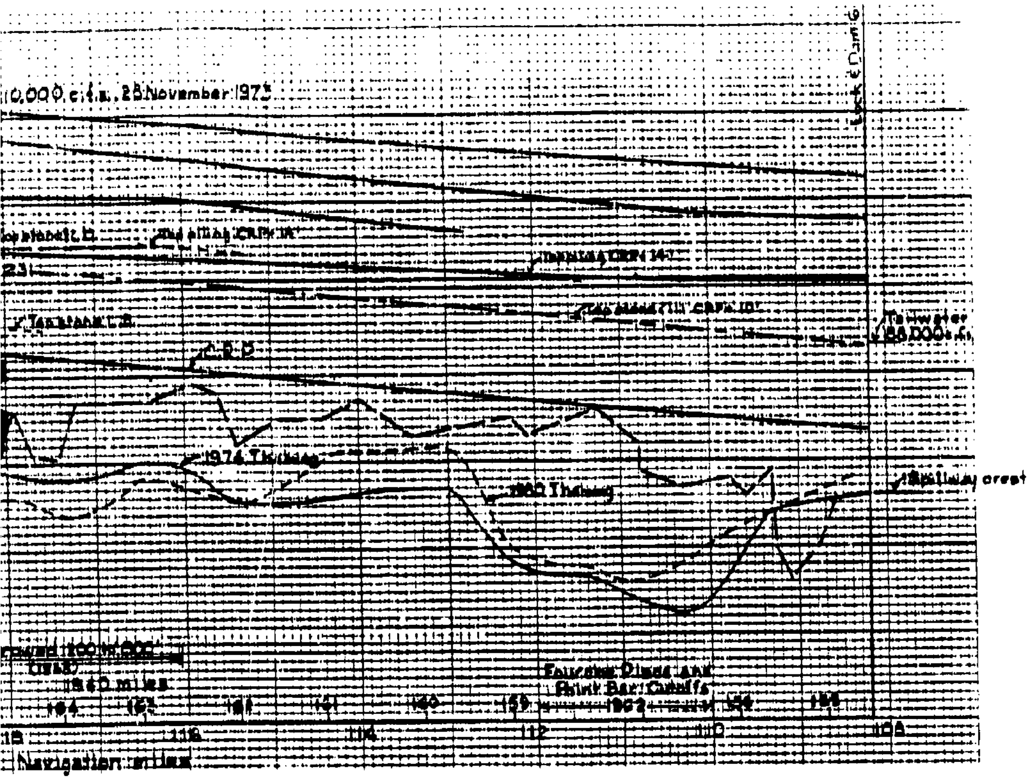
Arkansas River, Arkansas

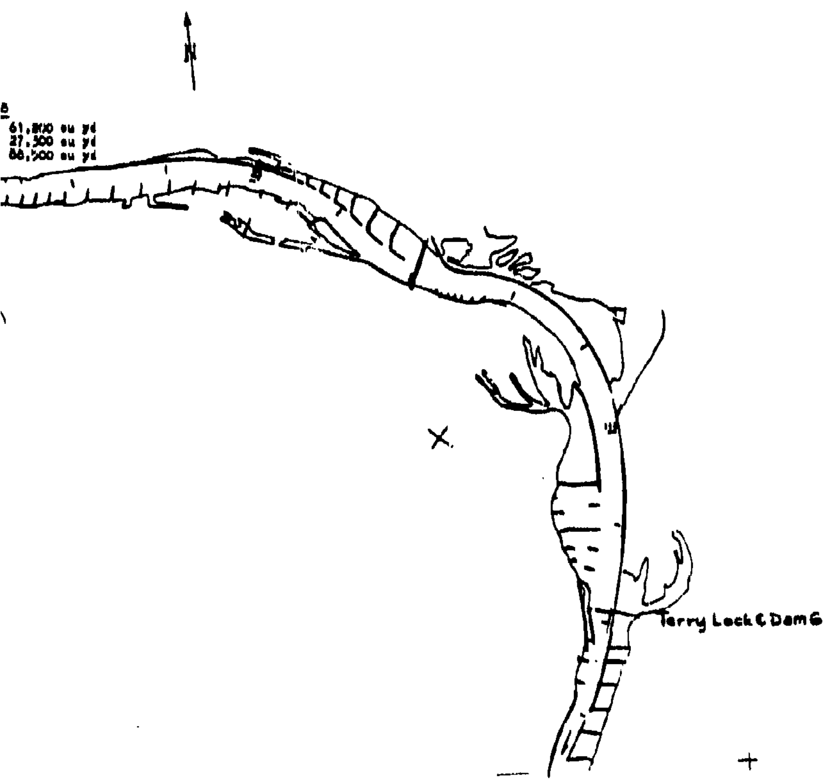
Layout and Profiles

Pool 7

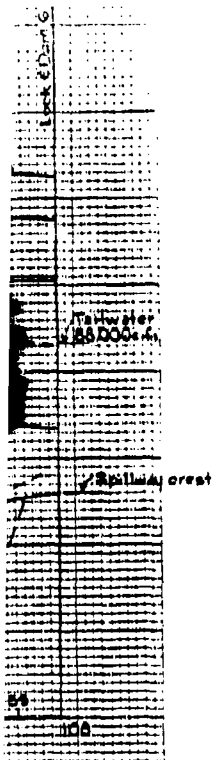
N. Mile 155.8 to N. Mile 125.4


460,000 c.f.s., 1 June 1957
Navigation miles
1974 Thalweg
Crest
Lock & Dam 7
Mile 124
Mile 122
Mile 120
Total 18,800 cu yd
Mile 118
Total 88,500 cu yd
Distance in 1000 feet
N


10,000 c.f.s., 25 November 1973
Lock & Dam 6
Tailwater
88,000 c.f.s.
1974 Thalweg
1960 Thalweg
Spillway crest
Fourche Dikes and
Point Bar Cutoffs
1962
1960 miles
118
116
114
112
110
108
Navigation miles


N
61,800 cu yd
27,300 cu yd
88,500 cu yd
Terry Lock & Dam 6

2



Arkansas River, Arkansas

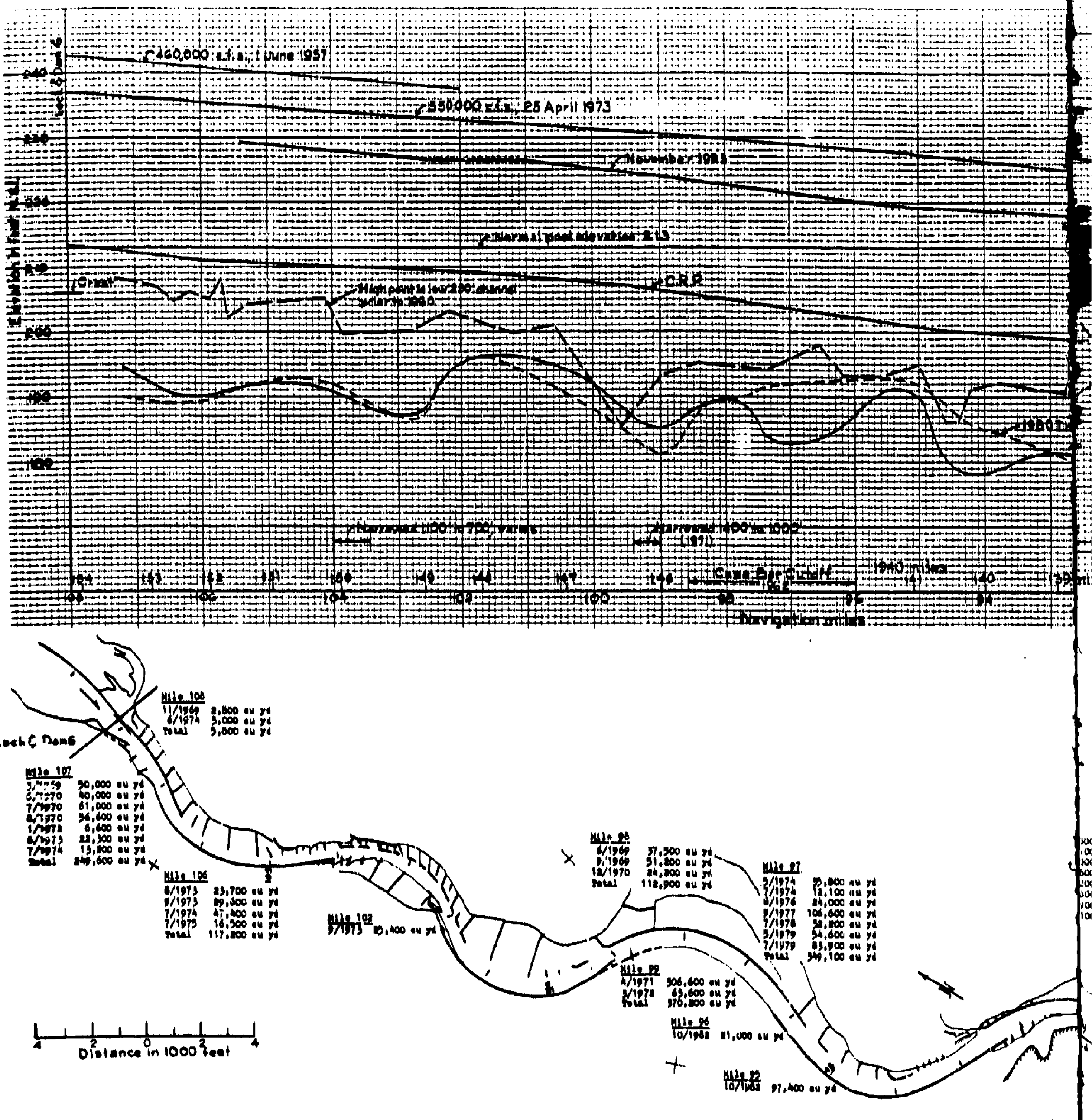
Layout and Profiles

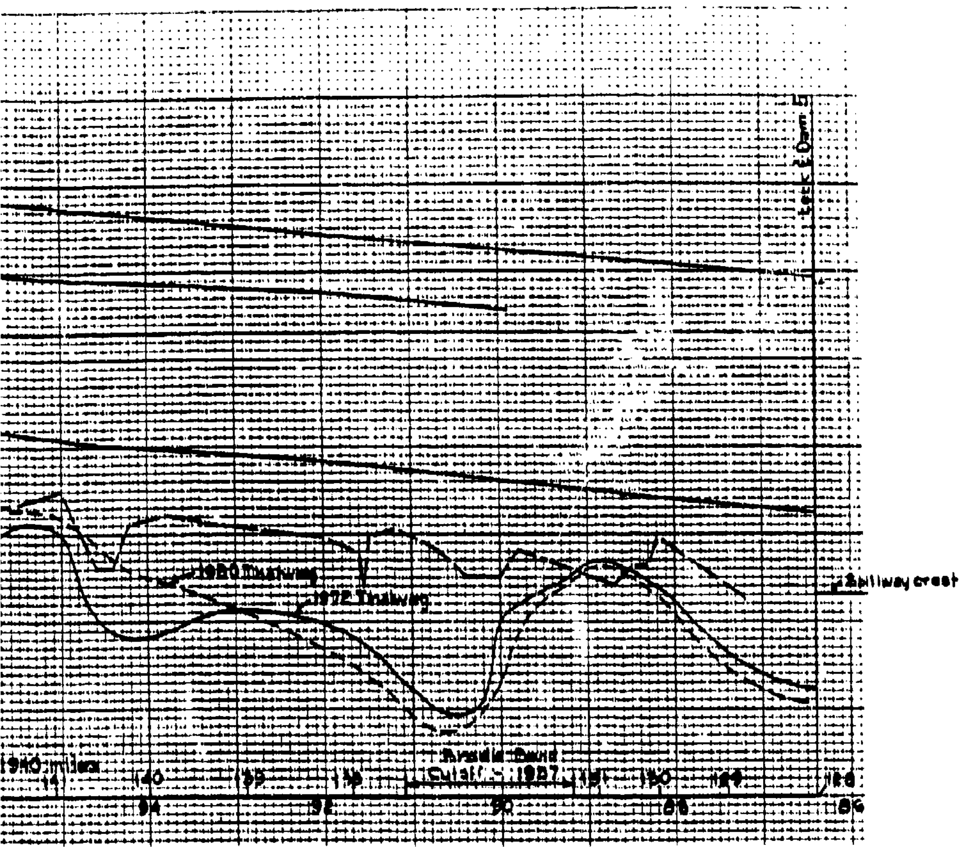
Pool 6

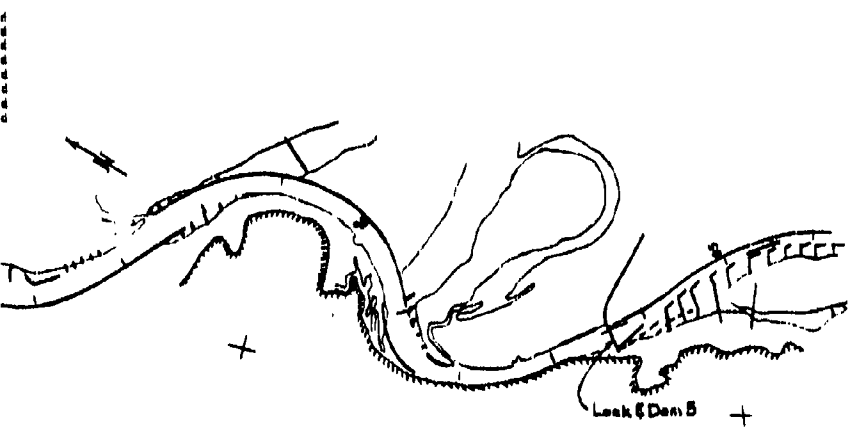
N. Mile 125.4 to N. Mile 108.2

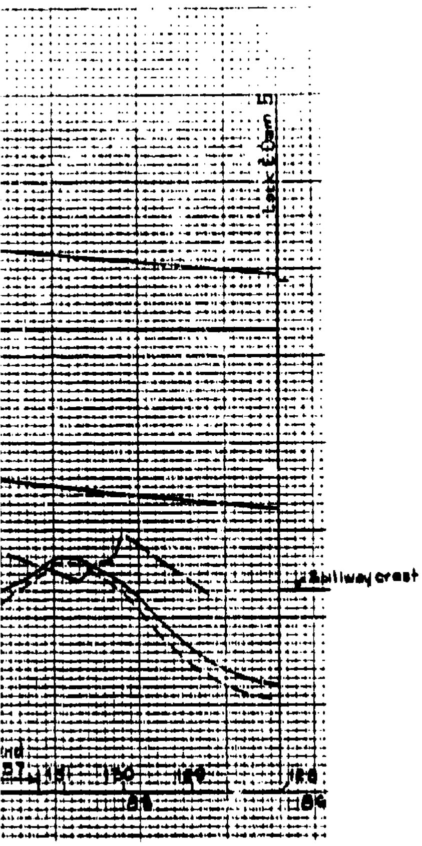
Figure 2-4

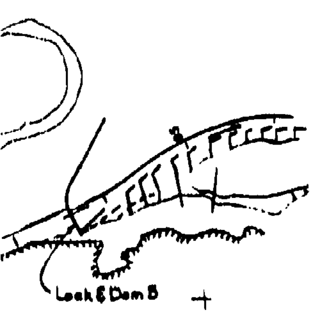
3


460,000 c.f.s., 1 June 1957
550,000 c.f.s., 25 April 1973
November 1983
Normal pool elevation 213
Crest
High point is low 200 channel prior to 1960
C.R.P.
Cases Bar Cutoff
Navigation miles
Lock & Dam 6
Mile 108
11/1969 2,800 cu yd
6/1974 3,000 cu yd
Total 5,800 cu yd
Mile 107
5/1969 50,000 cu yd
6/1970 40,000 cu yd
7/1970 61,000 cu yd
8/1970 56,600 cu yd
1/1972 6,600 cu yd
8/1973 22,300 cu yd
7/1974 13,800 cu yd
Total 249,600 cu yd
Mile 106
8/1973 23,700 cu yd
9/1973 29,300 cu yd
7/1974 47,400 cu yd
7/1975 16,500 cu yd
Total 117,800 cu yd
Mile 102
9/1973 25,400 cu yd
Mile 98
6/1969 37,500 cu yd
9/1969 51,800 cu yd
12/1970 24,800 cu yd
Total 112,900 cu yd
Mile 97
2/1974 55,800 cu yd
7/1974 12,100 cu yd
8/1976 24,000 cu yd
9/1977 106,600 cu yd
7/1978 38,200 cu yd
5/1979 54,600 cu yd
7/1979 83,900 cu yd
Total 349,100 cu yd
Mile 99
4/1971 306,600 cu yd
5/1972 63,600 cu yd
Total 370,200 cu yd
Mile 96
10/1982 21,000 cu yd
Mile 95
10/1982 97,400 cu yd
Distance in 1000 feet


Lock & Dam 5
1972 Thalweg
Spillway crest
140
94
92
90
186


Lock & Dam 5





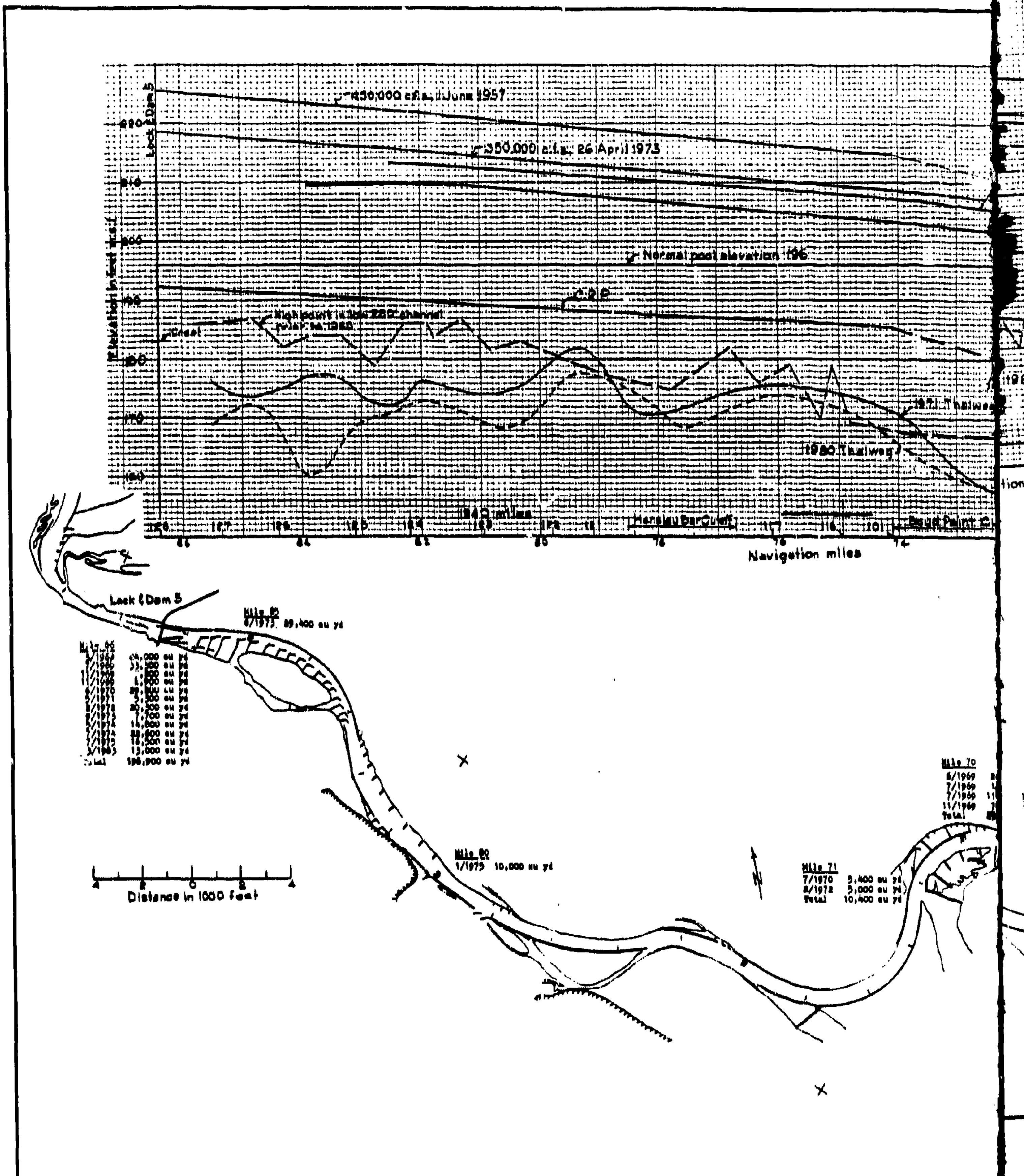
Arkansas River, Arkansas

Layout and Profiles

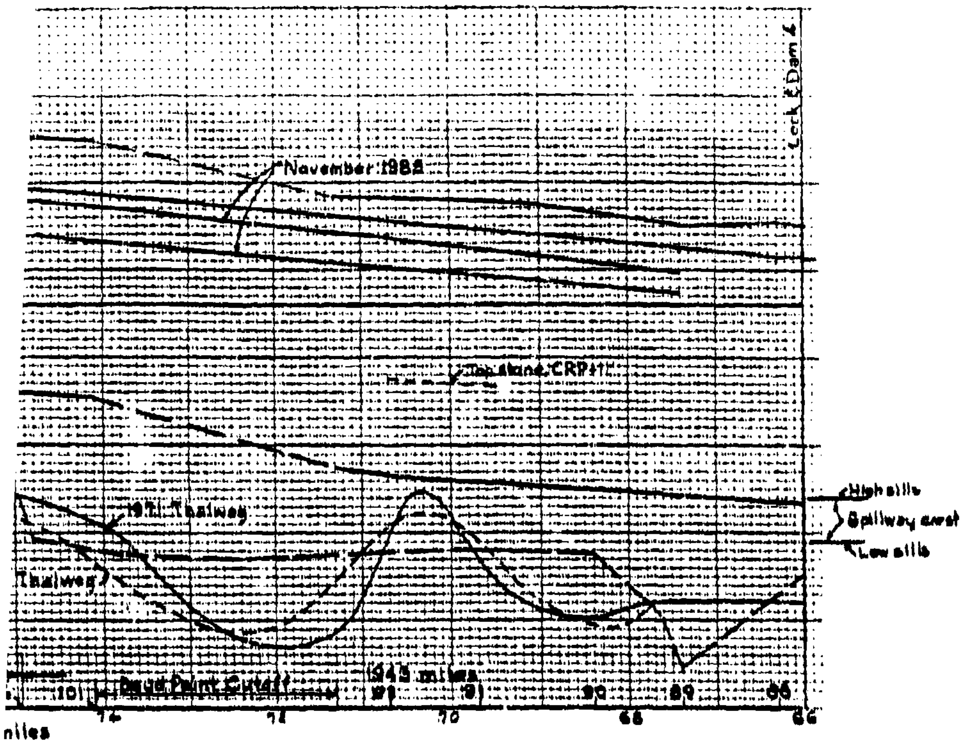
Pool 5

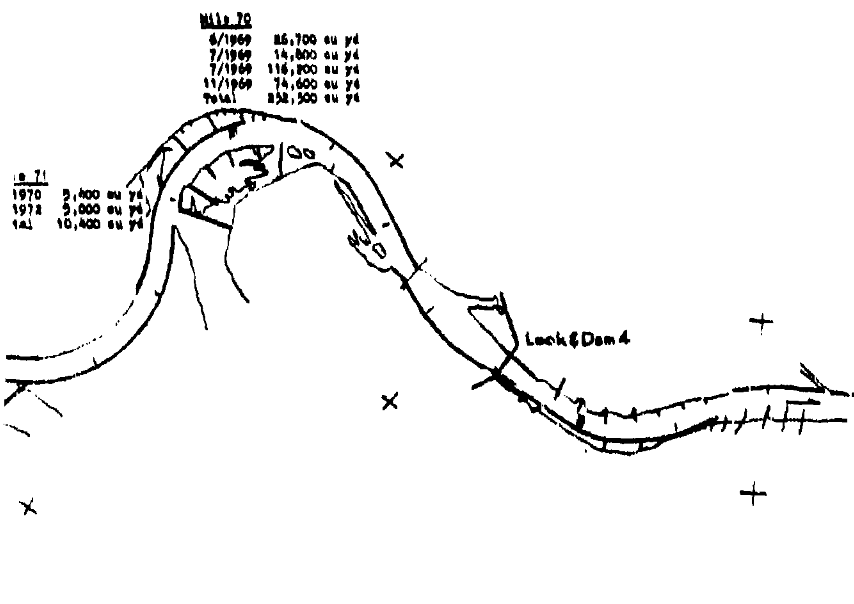
N. Mile 108.2 to N. Mile 86.4

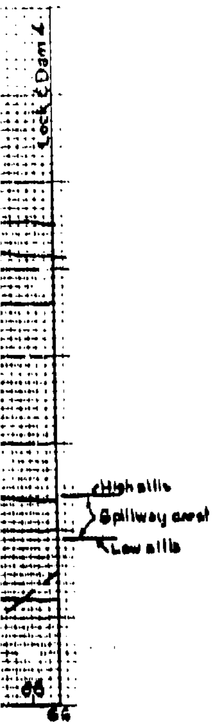
Figure 2-5



Lock & Dam 5
250,000 c.f.s., 11 June 1957
350,000 c.f.s., 26 April 1973
Normal pool elevation 196
C.R.P.
High point in low 1980 channel
Crest
1971 Thalweg
1980 Thalweg
Elevation in feet m.s.l.
Navigation miles
Lock & Dam 5
Mile 85
Mile 80
1/1975 10,000 cu yd
Mile 71
7/1970 5,400 cu yd
8/1972 5,000 cu yd
Total 10,400 cu yd
Mile 70
Distance in 1000 feet


Lock & Dam 4
High sills
Spillway crest
Low sills
Thalweg
miles


1970
Lock & Dam 4



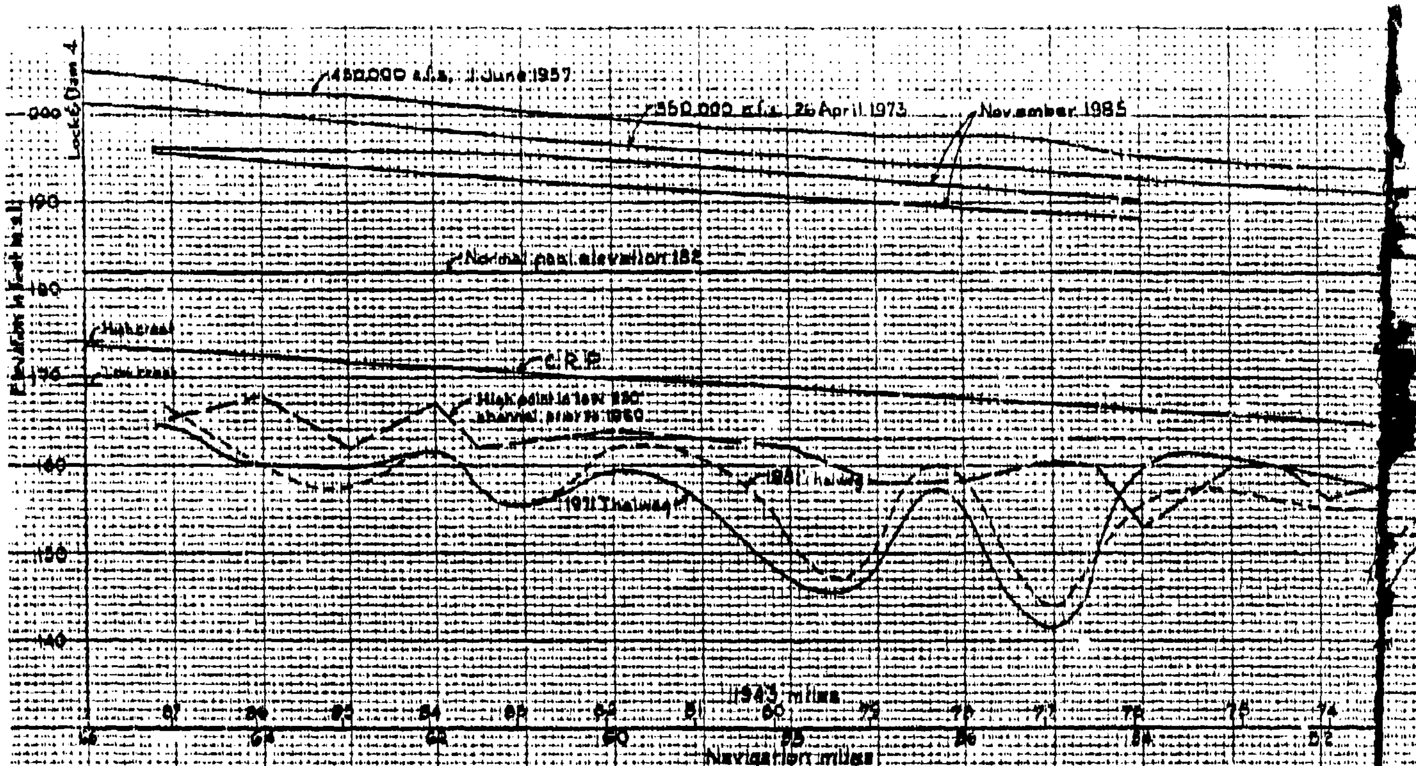
Arkansas River, Arkansas

Layout and Profiles

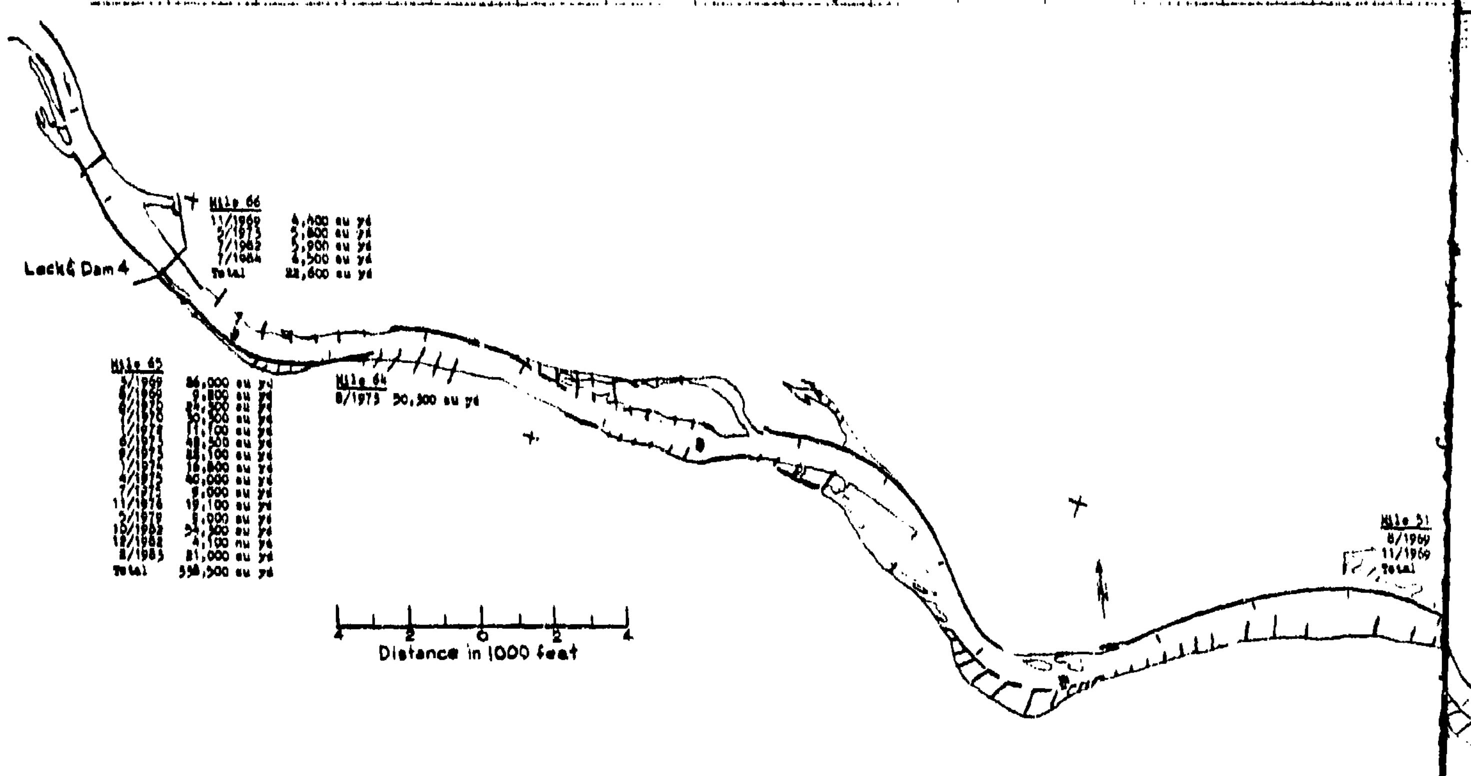
Pool 4

N. Mile 56.4 to N. Mile 66.0

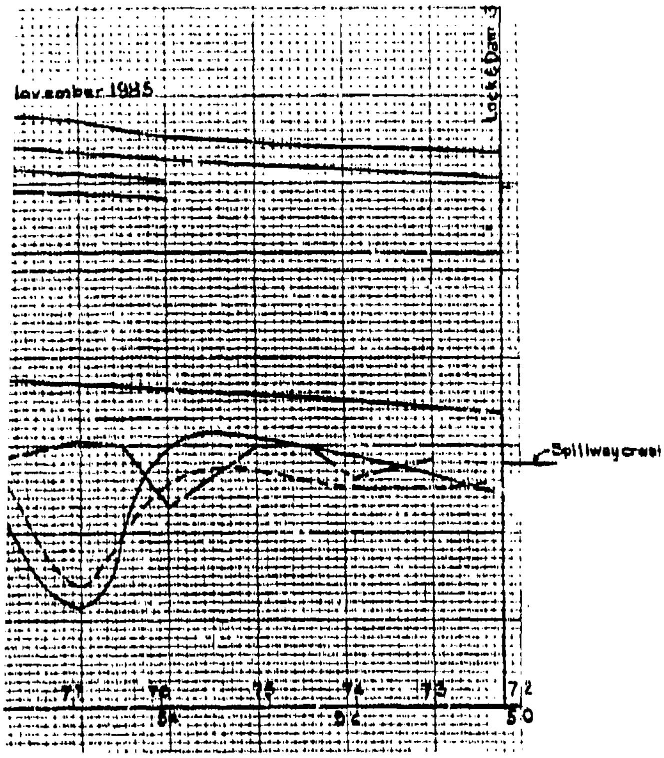
Figure 2-6

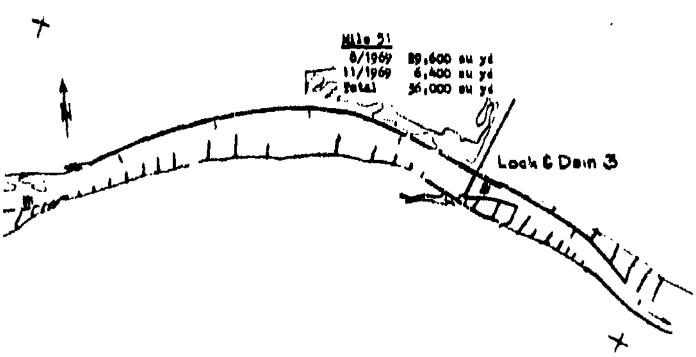


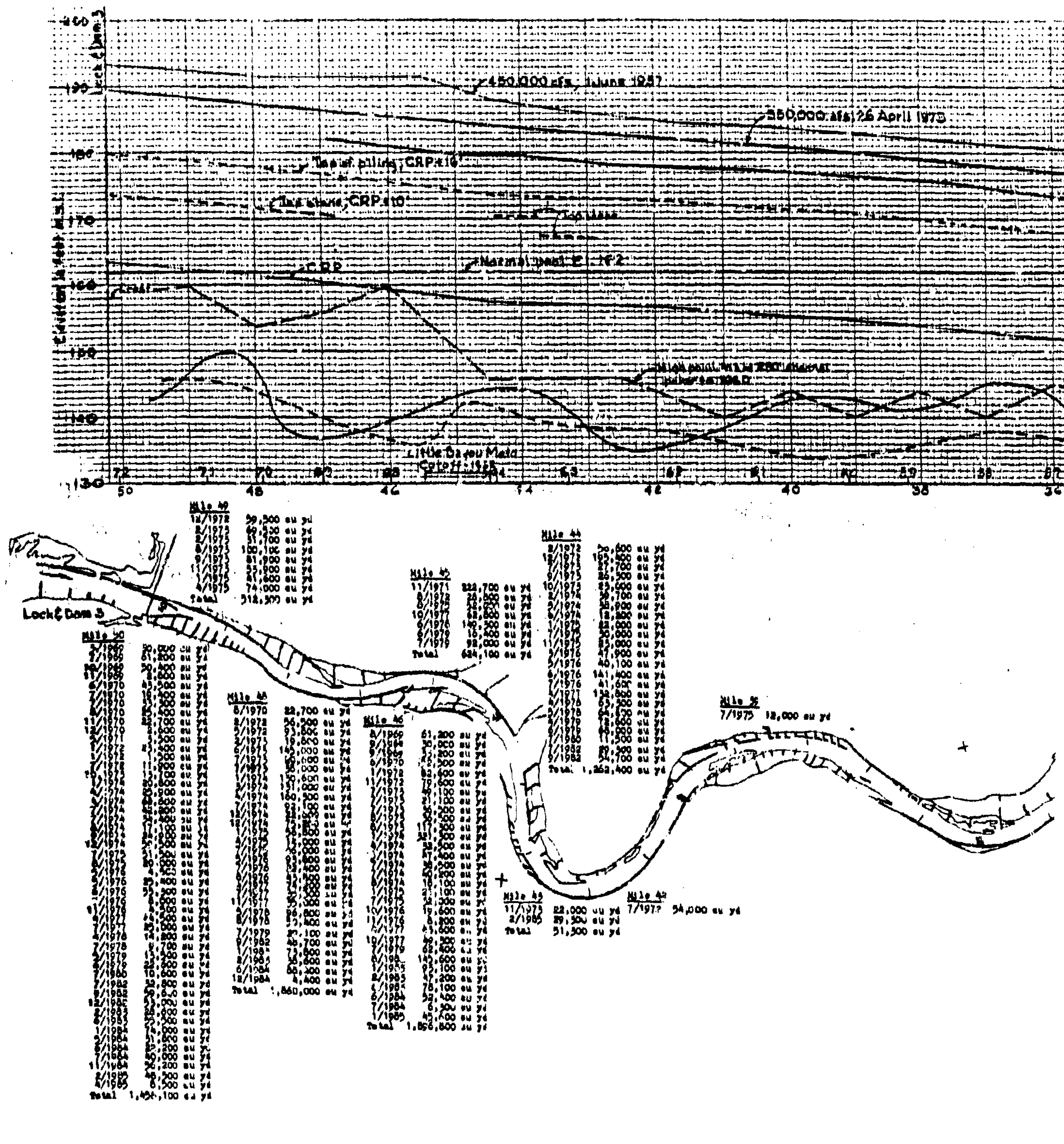
450,000 c.f.s., 1 June 1957
350,000 c.f.s. 26 April 1973
November 1985
Normal pool elevation 182
High pool
Low pool
C.R.P.
High points less 350' channel prior to 1960
1981 thalweg
1971 Thalweg
Lock & Dam 4
Elevation in feet m.s.l.
200
190
180
170
160
150
140
1983 miles
Navigation miles



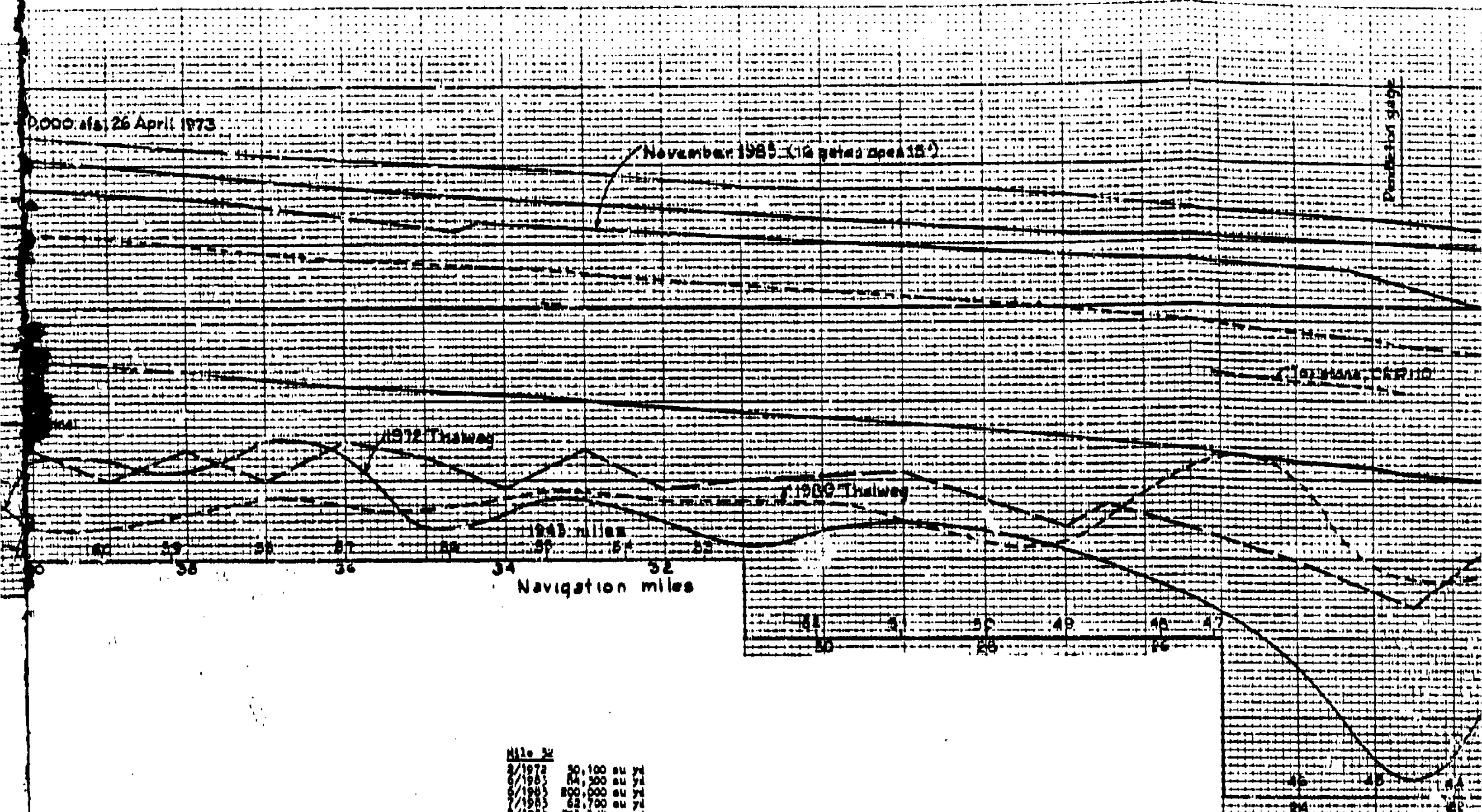
Lock & Dam 4
Mile 66
11/1969 4,400 cu yd
2/1973 2,800 cu yd
7/1982 5,900 cu yd
7/1984 6,500 cu yd
Total 22,600 cu yd
Mile 65
Total 558,500 cu yd
Mile 64
8/1973 50,300 cu yd
Mile 51
8/1969
11/1969
Total
Distance in 1000 feet
4
2
0
2
4


Lock & Dam 3
Spillway crest

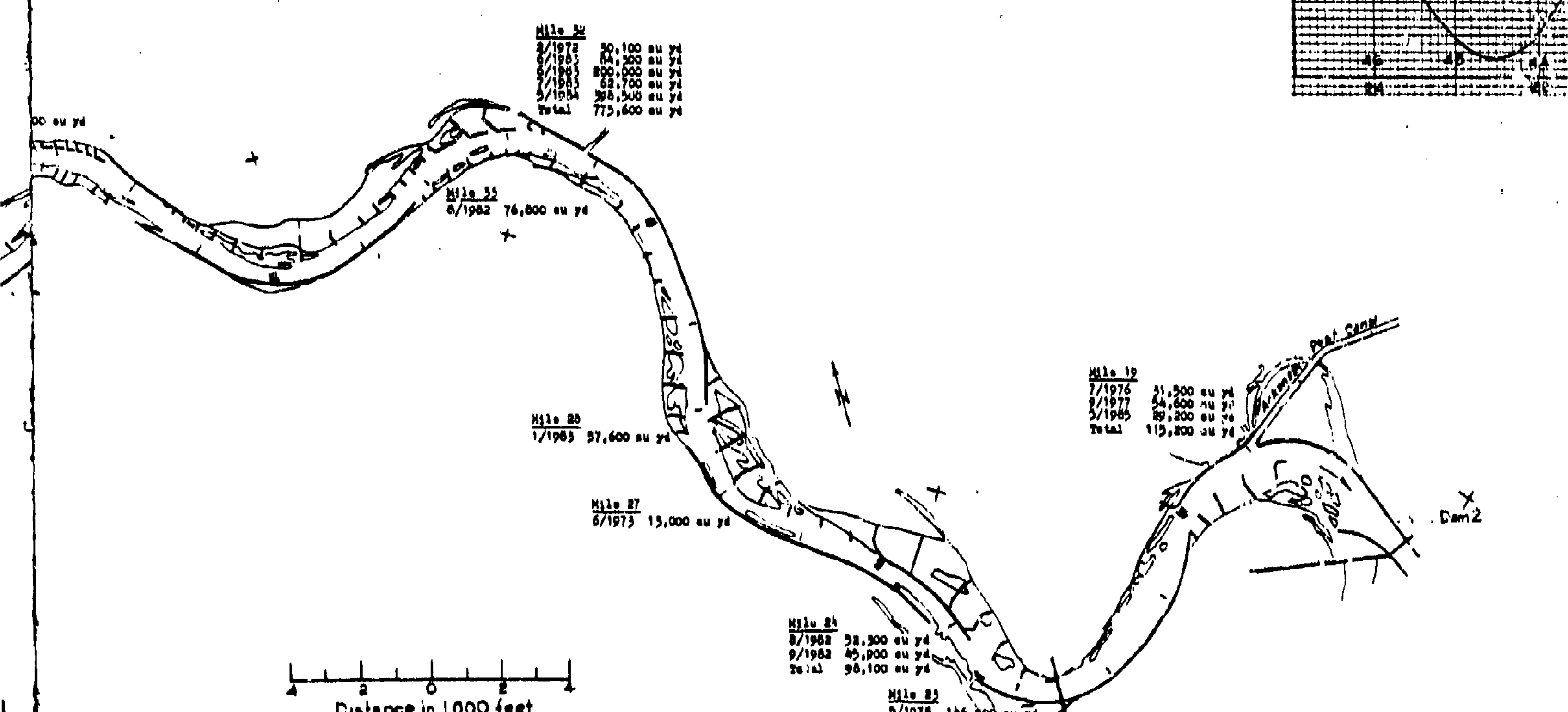

Lock & Dam 3



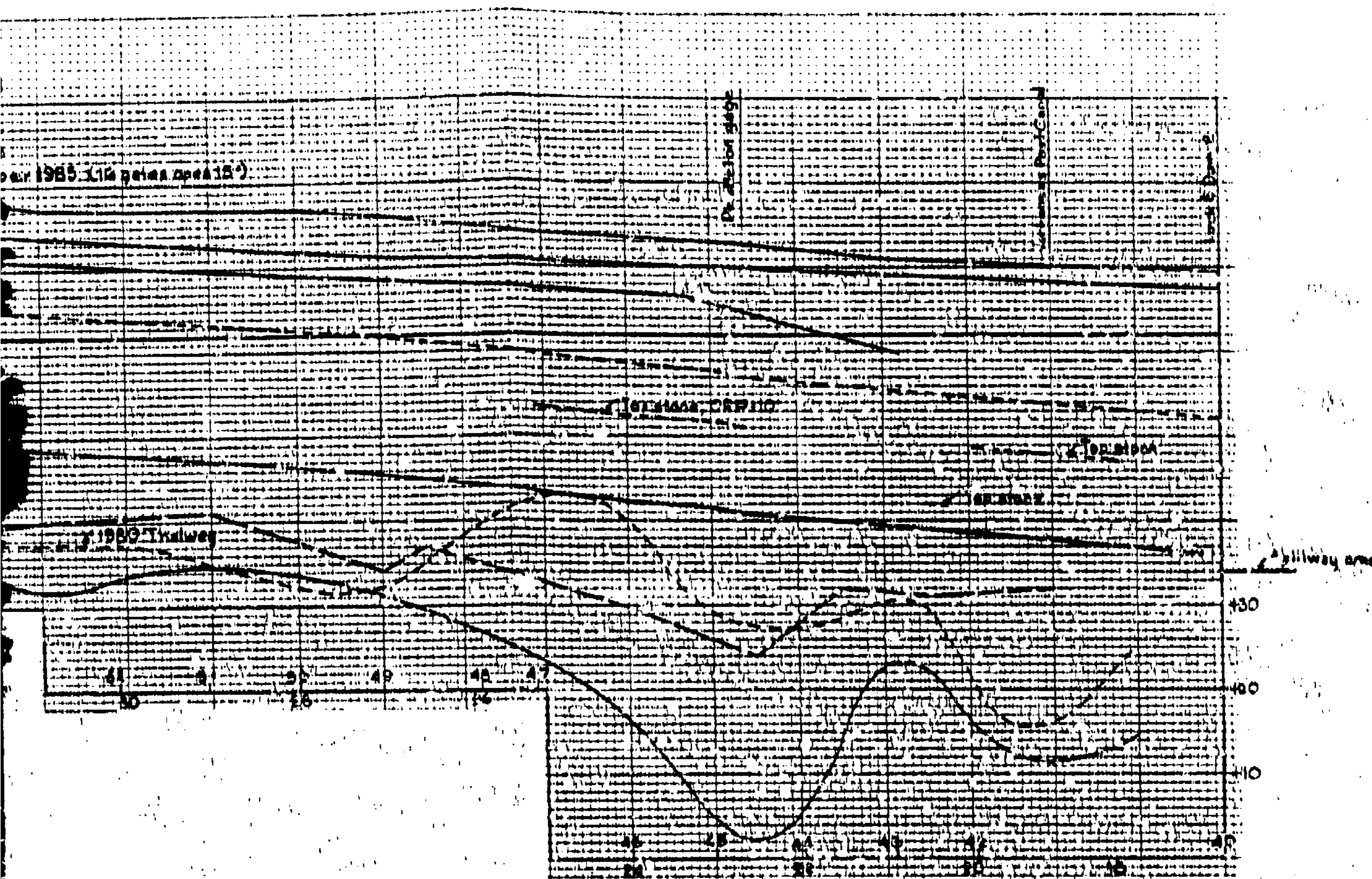
450,000 cfs, 1 June 1957
350,000 cfs, 26 April 1973
Top of piling, CRP+10
Top stone, CRP+10
Normal pool El. 172
CRP
Crest
Little Bayou Mato Cutoff 1958
Lock & Dam 3
Mile 49
Mile 50
Mile 48
Mile 46
Mile 45
Mile 44
Mile 43
Mile 42
Mile 36

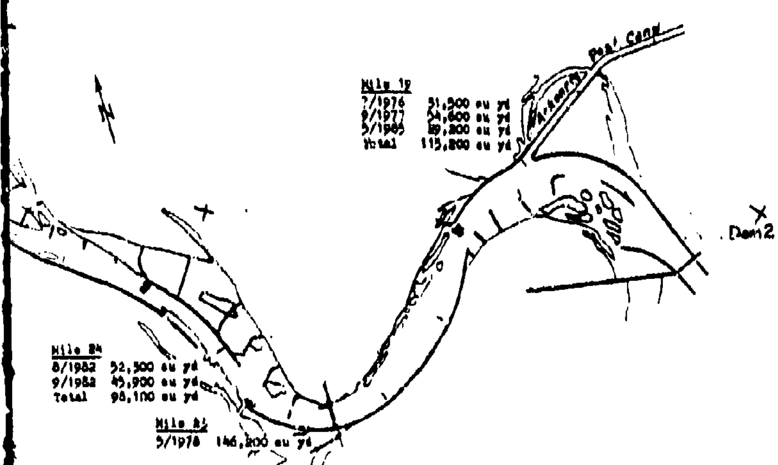


April 1973
November 1985 (16 gates open 15')
1972 Thalweg
1986 Thalweg
1985 miles
Navigation miles
Pendleton gage



Mile 19
7/1976 31,500 cu yd
Total 115,200 cu yd
Mile 28
1/1985 57,600 cu yd
Mile 27
6/1973 15,000 cu yd
Mile 24
8/1982 52,300 cu yd
9/1982 45,900 cu yd
Total 98,100 cu yd
Mile 23
5/1978 146,200 cu yd
Mile 33
8/1982 76,800 cu yd
Total 775,600 cu yd
Dam 2
Distance in 1000 feet





Arkansas River, Arkansas

Layout and Profiles

Pool 2

N. Mile 50.2 to N. Mile 17.0

Table 2-2

Years For Which Basic Date Were Available

Pool	Cross Sections	High Water Profile(1) Discharge (cfs)	Date	Bed Material Composition
9	1975, 79, 81	460,000	May 57	1971, 72, 74, 75
		330,000	Nov 73	
		83,700	Apr 78	
8	1975, 79, 80	460,000	May 57	1971, 75
		330,000	Nov 73	
		95,500	Apr 78	
7	1969, 75, 79, 80	460,000	May 57	1971, 75
		320,000	Nov 73	
		95,500	Apr 78	
6	1974, 75, 78, 80	460,000	Jun 57	1970, 72, 75
		310,000	Nov 73	
		88,000	Apr 78	
5	1972, 75, 78, 80	460,000	Jun 57	1970, 76
		350,000	Apr 73	
4	1971, 75, 79, 80	450,000	Jun 57	1972, 76
		350,000	Apr 73	
3	1971, 78, 80, 81	450,000	Jun 57	1972
		350,000	Apr 73	
2	1972, 78, 80	450,000	Jun 57	1972
		300,000	Nov 73	

(1) Profile data for the November 1985 flood (discharge in the order of 220,000 cfs) became available after basic studies were completed.

navigation depth. The profiles show the highest elevation in the deepest 250-foot wide preproject channel (from the latest surveys prior to 1960), thalweg elevations in various years under project conditions, and representative high-water profiles. Location and volume of maintenance dredging by years are shown.

It should be noted that all cutoffs from Dardanelle Dam to Dam 6 that significantly shortened the river were in place prior to the spring 1957 high-flow period; that all other major cutoffs downstream of Dam 3 were in place by the fall of 1962; that construction of Dardanelle Dam began in 1959 and the dam was closed in 1963; and that construction of downstream low-lift navigation dams began with Dams 1 and 2 in 1963 and proceeded upstream. By 1968 all locks and dams throughout the system in Arkansas and Oklahoma were under construction, and the entire project was completed and operational in December 1970. The sequence of dates is important because project construction was expedited in the mid-1960's and there was not sufficient time for the rectified channel in some reaches to become fully stabilized prior to closure of the navigation dams as originally planned.

The characteristics of Pools 9 through 3 (based on available data) and historical maintenance dredging in each pool are discussed in the following sections. Maintenance dredging is discussed in detail in Chapter 4. Open-river flow conditions are discussed further in Chapter 6.

3-2 POOL 9

Degradation - Pool 9 is a long pool (28.6 mi); only Pools 7 and 2 (30.5 mi and 33.2 mi, respectively, are longer, and other pools range in

length from 21.8 to 15.8 mi. From Dardanelle Dam (N Mile 205.5) to N Mile 198 the bed was dredged initially and degraded in the order of 15 to 20 feet between 1960 and 1975, and there was no additional degradation in the reach between 1975 and 1981, Figure 2-1. Degradation was to be expected because of: (1) trapping of sediment in Dardanelle reservoir, (2) construction works at the head of Pool 9 and (3) Point Bar and Holla Bend cutoffs that shortened the river by about seven miles. Verbal communication indicated there was 17 million cu yd of initial dredging downstream of Dardanelle Lock and Dam as a part of project construction; only minor maintenance dredging has been required immediately downstream of the lock. Top of rock from Dardanelle Dam to N Mile 193 is, at places, near the 1975-1981 thalweg. It is not clear from available data whether rock now controls the bed elevation in the upper half of Pool 9 or rock is below an armored bed.

In the ten miles downstream of the cutoffs (to N Mile 184) the bed appears to have lowered from a few feet to ten ft. The bed is generally little changed and is at about spillway crest level for six miles immediately above Dam 9 (N Mile 183-177); the spillway crest appears to control the bed elevation in that reach.

Contraction works have narrowed the river somewhat throughout the pool, and the bed has lowered perhaps about three feet as a result of contraction. After the project was in operation, additional narrowing was provided at several locations:

- In the relatively straight 3-mile reach below Dardanelle Dam (from 1,000 ft to 800 ft).

- In the long flat crossing at N Mile 196 (from 1,000 ft to 750 ft).

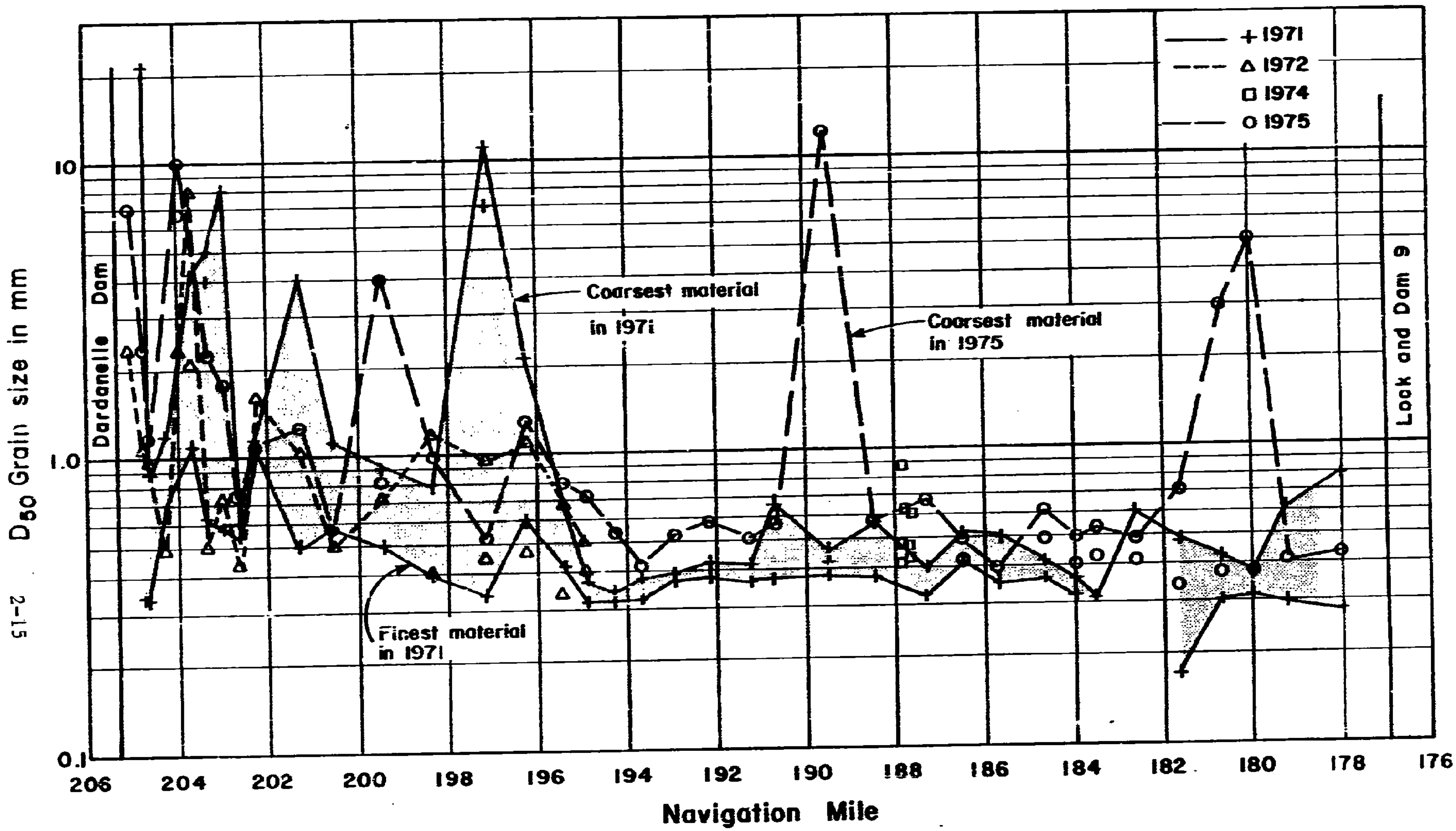
- In the flat crossing at N Mile 190 (from 1,200 ft to 800 ft).

- In the very flat bend from N Mile 189 to 197 (from 1,200 ft to 900 ft).

The pattern of bed lowering in Pool 9 supports the conclusion that the primary causes of degradation are the drastic reduction in sediment load in outflow from Dardanelle Dam, shortening the reach by cutoffs, and narrowing the channel.

Bed Material - Measured median grain size of bed material through Pool 9 for years 1971 through 1975 is shown in Figure 2-9. The data indicate that bed material becomes finer with distance downstream from Dardanelle Dam (from in the order of 3 mm at the head of the pool to 0.4 mm in the lower part of the pool). Material in the lower portion of the pool appears to have coarsened from about 0.4 to 0.5 mm between 1971 and 1975.

Water Surface Slope - The overall water surface profile of the November 1973 flow of 330,000 cfs is slightly less steep (0.84 ft per mile) than that of the 1957 flow of 460,000 cfs (0.88 ft per mile) under preimpoundment conditions, as is to be expected with degradation, and about the same as that of the 1985 flood (0.85 ft per mile). The flat water surface slope (0.49 ft per mile) for the April 1976 flow of 83,700 cfs was probably influenced by spillway gate control since the discharge is less than that for open river conditions at Dam 9.



Bed Material
Median Grain Size
Pool 9

Figure 2-9

Maintenance Dredging - There has been no maintenance dredging in the first ten miles below Dardanelle Dam except minor amounts at N Mile 205 in 1972 (30,600 cu yd) and N Mile 198 in 1984 (7,900 cu yd), Figure 2-1. There was considerable dredging (265,800 cu yd) in the 3-mile relatively straight reach (crossing) at the lower end of Holla Bend Cutoff at N Mile 194 in the 1970's, and a very minor amount at N Mile 193 in 1984. A total of 725,800 cu yd was dredged in the 1970's (76 percent prior to 1975) from the seven-mile reach between N Miles 189 and 182; the reach has very little sinuosity. Since 1975 there has been only a modest amount of dredging in Pool 9, mostly in the downstream portion of the pool. In 1982 and 1984, 68,000 cu yd of material was dredged at N Mile 186 and 42,200 cu yd in the long crossing at N Mile 181. The only other recent dredging in the pool was 7,900 cu yd in 1984 at N Mile 198 noted above. It should be noted that 1973 through 1975 were years of above average flow, as discussed further in Chapter 4.

Discussion - Pool 9 is a long pool (28.6 mi), and normal pool elevation in the downstream 17 miles of the pool is set below the construction reference plane, Figure 2-1, as discussed further in Section 6-2. Top of piling and top of stonefill in control structures are generally above normal pool elevation except in the immediate vicinity of Dam 9. The structures appear to effectively control flows within the main channel at discharges up to the range of 130,000 cfs (or for about 90 percent of the time).

The pool is bordered by rock escarpments at intervals along both the left and right banks, and the rectified channel alignment has very little

sinuosity in the upper 22 miles of the pool, with long and poorly defined crossing reaches.

It is likely that the good performance of structures in Pool 9 and the relatively modest maintenance dredging required in the past ten years are related to the significant decrease in sediment transport through the reach with Dardanelle Dam in operation and to the fact that open-river flow conditions prevail for discharges at or above 137,000 cfs, a flow that exists, on the average, five percent of the time, and that appears to have occurred in 17 of the 22 years following closure of Dardanelle Dam, as discussed in Chapter 6.

2-3 POOL 8

Degradation - Pool 8 is relatively short, about 21 miles long. Morrilton Cutoff, which was opened in 1960, is located about midway along the pool and shortened the river by about 5.2 miles, Figure 2-2. Initial dredging at the head of Pool 8, as a part of project construction, totaled 4.9 million cu yd. In the five miles below Dam 9 the thalweg lowered about 5 to 10 ft between 1960 and 1975. In the downstream 15 miles of Pool 8 bed change varied from section to section. The bed overall did not change significantly from 1960 to 1980 although at some places it might be several feet higher or lower. It is probable that the spillway crest exerts at least partial control on bed and water surface elevations through the downstream portion of Pool 8.

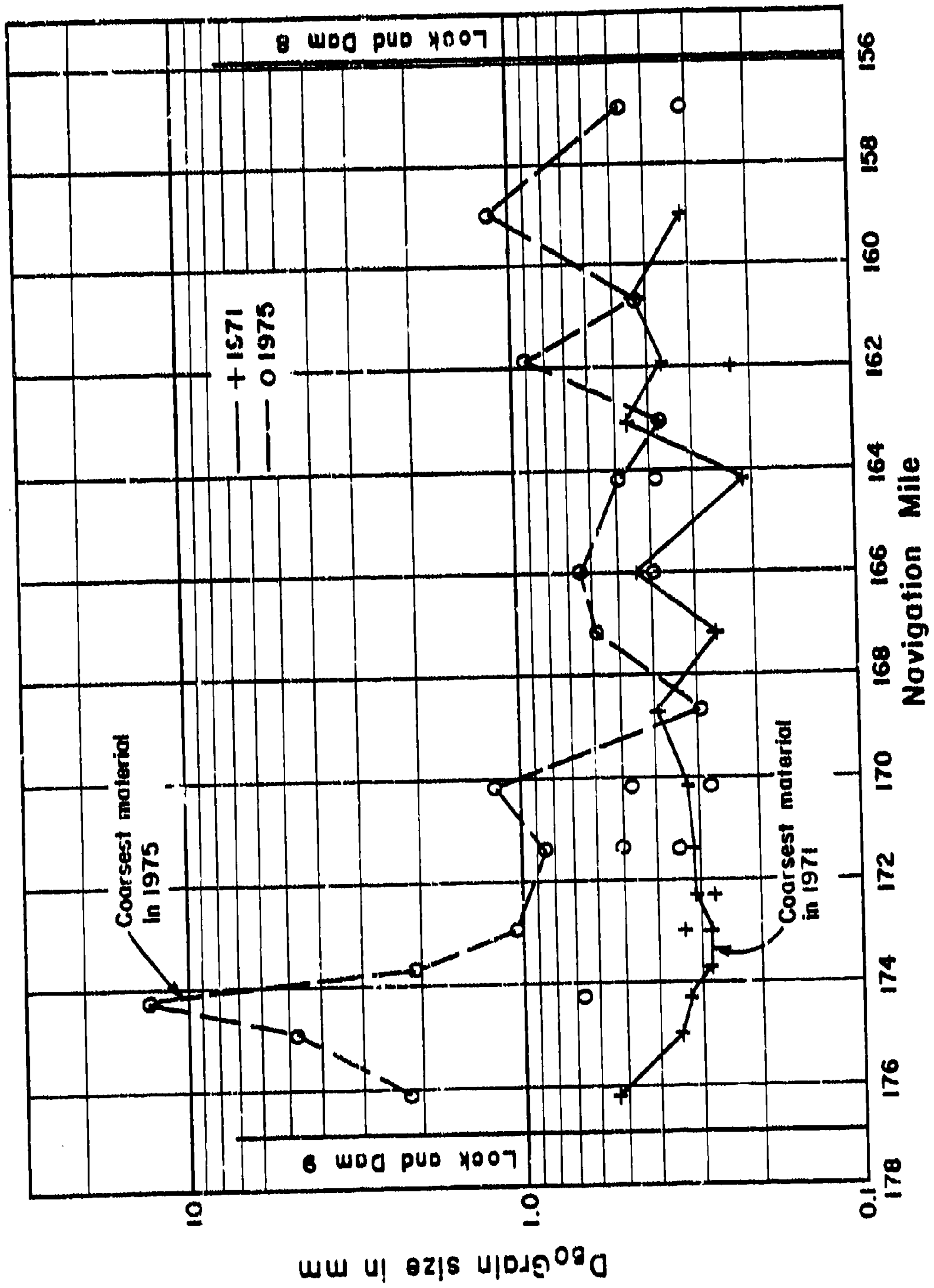
Structures narrow the river throughout most of the pool, but the initial narrowing was generally modest. After the project was in operation,

some reaches of river were narrowed further to as little as 600 ft, as follows:

- The flat crossing at N Mile 171 (from 1,200 ft to 600 ft).
- The long flat crossing at N Mile 167 (from 1,200 ft to 750 ft).
- The upper lock approach at N Mile 156.8 (from 1,200 ft to 800 ft).

Bed Material - Measured median grain size of bed material through Pool 8 in 1971 and 1975 is shown in Figure 2-10. The data indicate that in 1971 median grain size throughout the pool was about the same, 0.35 mm. Data from 1975, however, indicate a coarsening of bed material throughout the pool, as compared to 1971, with a D_{50} grain size in the order of 4 mm at the head of the pool decreasing to about 0.5 to 1.0 mm in the downstream portion of the pool.

Water Surface Slope - The water surface slope (0.78 ft per mile) through the pool for a discharge of 320,000 cfs in November 1973 was less than for a discharge of 460,000 cfs in 1957 (0.9 ft per mile). The 1978 profile for a discharge of 95,500 cfs was about parallel to that of the 1973 flood, with a slope of 0.74 per mile. Since Dam 8 spillway gates are fully open at a discharge of about 80,000 cfs, the 1973 profile (as well as the 1978 profile) appears to reflect open-river project conditions. Slope of the 1985 profile was also about 0.78 ft per mile, the same as for the November 1973 flood.



Bed Material
Median Grain Size
Pool 8

Figure 2-10

Maintenance Dredging - All dredging in Pool 8 was done prior to 1978 and was almost entirely in the upstream half of the pool, totaling about 2.8 million cu yd over eight years.

Dredging in the upper half of the pool amounts to about 5 ft over a 300-ft width, and probably is about equal to the lowering that would have naturally occurred eventually due to degradation and long-contraction scour. The fact that this dredged material was disposed of outside the main channel could account for the lack of need to dredge in the lower half of Pool 8.

The considerable dredging in the 1970-1976 period indicates Pool 9 was supplying a significant load throughout those years. Lack of need for maintenance since that time would seem to indicate that sediment supply and transport capacity were in balance in the later 70's and early 80's. Whether the head of Pool 8 is now degrading because of a meager sediment supply is not known.

In the early 1970's, the channel was narrowed from 1,200 to 750 ft over a short reach to increase sinuosity at N Mile 167 just upstream of Morrilton cutoff and vane dikes were used at N Mile 172-171 to narrow the crossing.

Discussion - Pool 8 is a short pool (21 mi), and normal pool elevation is set higher than Pool 9 with respect to the construction reference plane, Figure 2-2. Top of stone in control structures is generally above normal pool elevation except immediately upstream from the dam. Thus the structures effectively control flows within the main channel about 90 percent of the time, as in Pool 9. The pool is bordered by rock escarpments at intervals along both the left and right banks, and the

rectified channel has little sinuosity in the reaches N Mile 174-170 and 168-160.

It is likely that the good performance of structures in Pool 8 and the lack of need for maintenance dredging in the 1978 - 1984 period are related to the decrease in sediment transport through the reach due to Dardanelle Dam, to removal of sediment by dredging from the main channel in Pool 9, and to the fact that open-river conditions prevail at discharges of 80,000 cfs and higher, conditions that exist on the average about 15 percent of the time and can be expected to occur annually. A mean daily discharge of 80,000 cfs or higher appears to have occurred in 20 of the 22 years following closure of Dardanelle Dam, as discussed in Chapter 6.

2-4 POOL 7

Degradation - Initial dredging of 8.3 million cu yd, as a part of project construction, and contraction works lowered the bed from 5 to 10 ft for about five miles through the long flat bend at the head of the pool between 1960 and 1969. Post-project surveys in 1969, 1975, and 1980 do not indicate significant trends in change of thalweg elevation throughout the pool in that period, Figure 2-3.

Lack of the need for maintenance dredging since 1976 in the first five miles at the head end of the pool may mean that the bed has degraded to a stable elevation in that reach for present conditions. The bed slope is slightly adverse downstream of Dam 8 for about 6 miles and then horizontal for about 10 miles, Figure 2-3. The bed will probably continue to degrade,

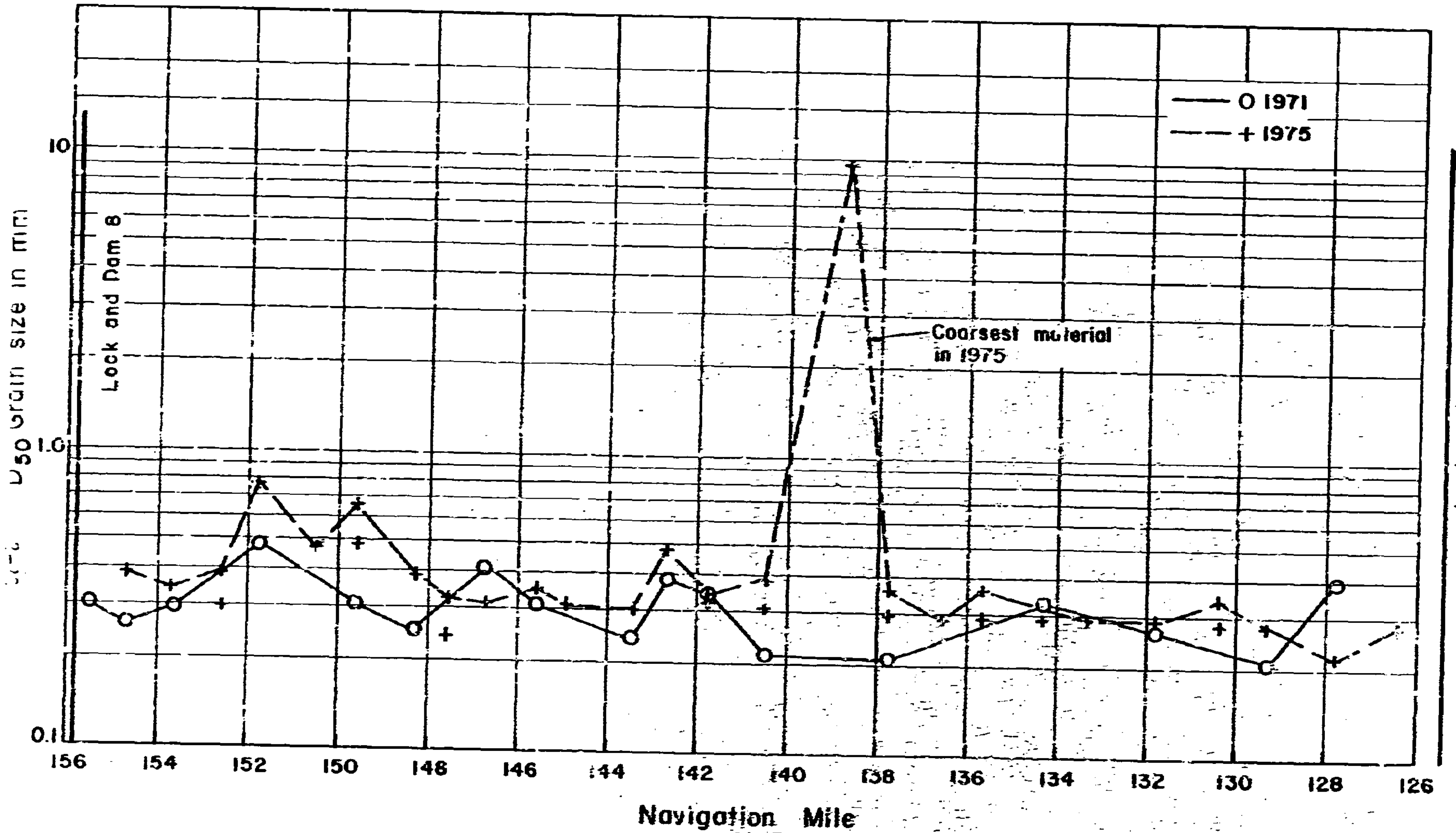
fluctuating between scour and fill with changing flow rates and pool operation.

In the early 1970's, after the navigation project became operational, much of the channel in the upper half (14 miles) of Pool 7 was narrowed to 600 or 700 ft, as follows:

- Immediately downstream from Lock 8 (from 1,200 ft to 700 ft).
- In the long flat crossing and long radius bend between N Miles 151 and 147.5 (from 1,100 ft to 600 ft).
- In the flat bend between N Miles 147.5 and 145 (from 1,000 ft to 700 ft).
- In the very long crossing at N Mile 142 (from 1,400 ft to 700 ft).

Bed Material - Measured median grain size of bed material through Pool 7 in 1971 and 1975 is shown in Figure 2-11. The data do not indicate any trend in coarsening of bed material during that time except perhaps a slight coarsening near the head of the pool. Median grain size generally becomes finer through the pool, from about 0.6 mm near the head of the pool to about 0.25 at Dam 7.

Water Surface Slope - The overall water surface slope through the pool for 320,000 cfs in 1978 (0.71 ft per mile) is about the same as for 460,000 cfs through the reach in 1957 (0.72 ft per mile) prior to construction of the project. Water surface slopes in this reach were not uniform under preimpoundment or project conditions. In the upstream portion of Pool 7, the 1978 water surface profile for 95,500 cfs was about parallel



Bed Material
Median Grain Size
Pool 7

Figure 3-11

to the profile for 320,000 cfs, but the Dam 7 spillway crest controlled the water surface elevation in the downstream half of the pool. Open-river conditions do not occur in Pool 7 below flows in the order of 190,000 to 200,000 cfs. The slope for the 1985 profile was somewhat steeper, about 0.80 ft per mile.

Maintenance Dredging - About 2.8 million cu yd were dredged from the head end of Pool 7 in the 1970's and another 0.34 in the early 1980's. Most dredging was in the upper 10 miles of the pool. It appears that a wedge of bed material has been removed, with the apex ten miles downstream of Dam 8; the bed slope was slightly adverse in this reach in 1980. However, dredging was not in that same wedge pattern; by far the heaviest dredging was three to nine miles downstream from Dam 8.

As a rough approximation, the quantity of dredged material was about equal to the bed lowering, but the difference between the dredging and lowering patterns would indicate active sediment transport and sediment supply through Dam 8 in the 1970's. The continued need for dredging in the 1980's in the reach three to nine miles downstream from Dam 8 could mean that degradation due to sediment being trapped behind Dardanelle Dam had not reached Pool 7 to any significant degree by 1984.

Discussion - Pool 7 is 30.5 miles long, the second longest pool (after Pool 2) downstream of Dardanelle Dam, and there were no cutoffs in the reach. The pool is set below the construction reference plane at the upper end, Figure 6-1, and is one of the shallowest pools, with an average depth of 8.8 ft at normal pool level. However, because it is long, depths are

adequate in the lower portion of the pool. Open-river conditions begin at an elevation that is high with respect to top of control structures (at a flow of about 190,000 cfs) so that rock in control structures is submerged about 10 ft when open-river flow begins.

It is not clear hydraulically why eight of the 14 miles at the head end of Pool 7 had to be narrowed to 600 or 700 ft in the early 1970's, and requirements for continued maintenance dredging through the early 1980's suggest that even such extreme narrowing was not effective in transporting sediment through the pool. Maintenance dredging in Pool 7 through 1984 totaled 17.3 percent of all maintenance dredging in Pools 9 through 2. Possible reasons may be the relative frequency of open-river flow conditions through Pools 9 and 3, as compared to Pool 7, and the fact that stone in control structures in Pool 7 is submerged about 10 ft when open-river flow begins, as discussed in Chapter 6.

Pool 7 is bordered by rock escarpments at several locations, and while the upper reach of the pool has fairly good sinuosity, crossings in the upper half of the pool are several miles long. Storage capacity of Pool 7 at normal pool elevation is 86,000 ac ft, as compared to 56,000 ac ft in Pool 9 and 32 000 ac ft in Pool 8.

Because Pool 7 is gate-controlled up to flows of 190 000 to 200,000 cfs, open-river conditions through the pool can be expected to occur on the average only about one percent of the time and appear to have occurred in only 9 of the 22 years following closure of Dardanelle Dam, as discussed in Chapter 6. With spillway gates at Dam 8 fully open in 20 years of the

22-year period and good sediment transport through Pool 8 and into Pool 7, the longer detention time in Pool 7 may make it an effective sediment trap. With Pool 7 controlled for all but the largest flows and open-river conditions occurring infrequently, deposition would be expected in the upper half of the pool, and the area of heaviest dredging may be a backwater area where sands and any gravels that had deposited were later dredged out. In this case, only fine material would reach the lower end of the pool, and this has not deposited to any great extent. It may also be that degradation due to Dardanelle Dam was effective in lowering the bed in Pools 9 and 8 to provide adequate navigation depth but that such degradation has not yet been effective in Pool 7. In such a case, it would be necesary to rely on long-contraction scour to achieve greater depth, and a narrower channel would be needed in Pool 7 than in Pools 9 and 8.

2-5 POOL 6

Degradation - Available data indicate the bed has lowered about 10 to 20 ft immediately below Dam 7 where the channel was contracted and realigned and that at N Miles 112-110 through Fourche Place cutoff the channel bed is now 10 to 15 ft lower than in the old bend, Figure 2-4. There has been some lowering in the middle reach of the pool where the channel was narrowed in 1968 from 1,200 to 800 ft between N Miles 119 and 116 to guide navigation through the numerous bridges in the reach. In that stretch the 1980 bed profile was several feet below the 1974 profile and 5 to 10 ft below 1960

conditions, indicating that long-contraction scour occured. The bed in the downstream third of the pool was a few feet higher in 1980 than in 1974.

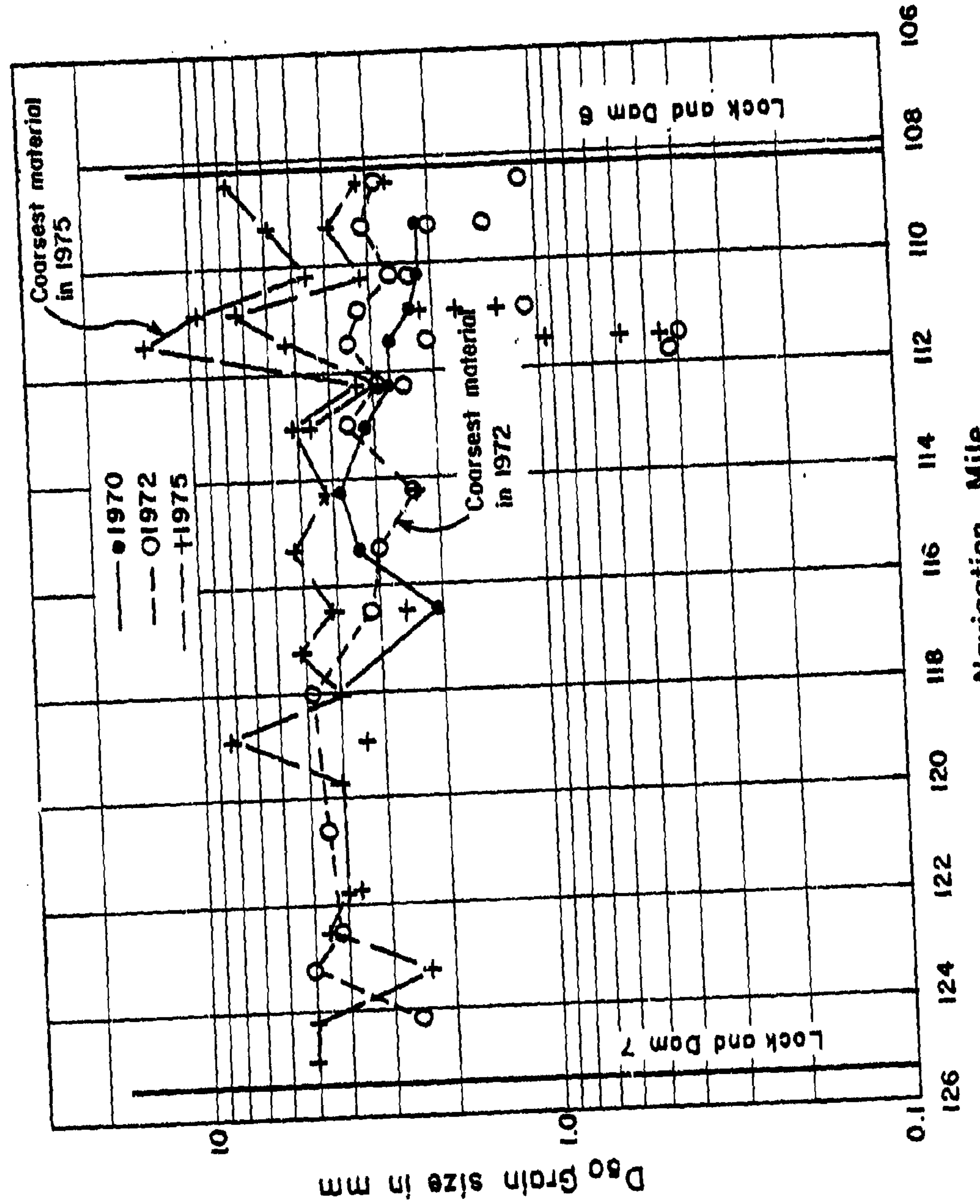
Bed Material - Data on median grain size distribution shown in Figure 2-12 indicate that there was some coarsening of the bed material between 1970 and 1975 in the downstream half of the pool. In the early 1970's median grain size was about 0.5 mm at the head of the pool, decreasing to about 0.25 mm at Dam 6. In 1975, the D_{50} size was about 0.5 mm for about 13 miles downstream from Dam 7, but was significantly coarser (in the range of 0.7 mm) in the next four miles downstream to Dam 6.

Water Surface Slope - Opening of Fourche Place and Willow Bar cutoffs in 1962 in the lower end of Pool 6 shortened the reach by about 1.5 miles. The water surface slope for the 1957 flow of 460,000 cfs through the reach was 0.85 ft per mile, somewhat flatter than the profile for the 1973 flow of 310,000 cfs of 1.05 ft per mile, but about the same as the profile for the 1985 flood (0.83 ft per mile).

Maintenance Dredging - Maintenance dredging was required in Pool 6 only in the early years of operation (1970-1973) and totaled only 169,000 cu yd. Dredging was required at only three locations:

- In the lower approach to Lock 7.
- In the crossing at N Mile 122.
- In the straight reach through numerous bridges at N Mile 118.

Since 1973, the two crossings at the head of the pool have performed well as designed.



Bed Material
Median Grain Size
Pool 6

Figure 2-12

Discussion - Pool 6, with a length of 17.3 miles, is one of the two shortest pools downstream of Dardanelle Dam and has an average depth at normal pool of 10.6 ft. The pool is set high with respect to the construction reference plane, Figure 6-1, but because the pool is short, top of piling in control structures is above or at normal pool elevation throughout the pool; thus the structures are effective in controlling flows up to about 100,000 cfs within the main channel.

Storage capacity at normal pool elevation is 50,000 ac ft, and flows are gate-controlled up to a discharge of about 155,000 cfs. Open-river conditions can be expected to prevail on the average about three percent of the time and appear to have occurred in 13 of the 22 years following closure of Dardanelle Dam, as discussed in Chapter 6.

The upper portion of Pool 6 has good sinuosity, and crossings are short. Little Rock District considers the downstream approach to Lock 7 to be the best in the system, and Pool 6 to be one of the two best pools in terms of effectiveness of the structures in providing reliable navigable depths.

It is likely that the good performance of structures in Pool 6 and the lack of need for maintenance dredging in the 1974-1984 period are related to the short length and relatively deep depth of the pool, the fact that open-river conditions occur with relative frequency, and to the decrease in sediment transport through the reach with Dardanelle Dam in operation.

2-6 POOL 5

Degradation - There has been significant lowering of the bed throughout Pool 5 between preproject conditions prior to 1960 and 1972-1980 conditions, Figure 2-5. The spillway crest elevation for Dam 6 was set at elevation 206 which was about the 1960 bed elevation. In 1980 the thalweg at the head of Pool 5 (immediately downstream of Dam 6) was about 10 ft below the Dam 6 spillway crest which is at about upstream bed level. The 1980 thalweg profile throughout the pool is not much different from the 1972 profile, a little higher at N Mile 98-93 and a little lower elsewhere. The crossings at N Miles 104 and 99 were narrowed after the project became operational from 1,100 to 700 and from 1,400 to 1,000 ft, respectively, over relatively short reaches.

Brodie Bend cutoff opened in April 1957 and enlarged significantly during the April 1957 flood, shortening the river by 5.55 miles. Case Bar cutoff opened in 1962, shortening the river an additional 2.8 miles. Brodie Bend cutoff is located about four miles upstream of Dam 5, and Case Bar cutoff is at about the midpoint of Pool 5. Preproject bed slope through the reach was about 0.8 ft per mile. In contrast, the 1972 and 1980 thalweg profiles are almost flat at an average elevation of about 193 from Dam 6 for 11 miles downstream to N Mile 97, and the average slope from N Mile 97 to Dam 5 is about 1.6 ft per mile.

Bed Material - Data on bed material composition in this pool are limited, but 1976 data indicate the D_{50} of the bed material appears to

decrease somewhat in a downstream direction. Median grain size at the head of the pool is about 0.6 mm, and it is about 0.3 mm just upstream from Dam 5, Figure 2-13.

Water Surface Slope - Water surface drop through the reach was 17 ft for the 1957 flow of 460,000 cfs, and the slope was 0.57 ft per mile. Brodie Bend and Case Bar cutoffs shortened the river 8.35 miles between 1957 and 1962, and the water surface slope in 1973 was 0.8 ft per mile for a discharge of 350,000 cfs, as would be expected with the degradation that had occurred. The slope of the 1985 profile was somewhat steeper, about 0.90 ft per mile.

Maintenance Dredging - There was considerable maintenance dredging (829,000 cu yd) in the upstream half of the pool prior to 1975, but no dredging has been required upstream of N Mile 98 since then. In the late 1970's and in 1982 dredging was required only in the long crossing at N Miles 97-95 (420,000 cu yd).

Discussion - Pool 5 is 21.8 miles long, but is set lower than Pool 6 with respect to the construction reference plane, Figure 6-1 . It is one of the three shallowest pools downstream of Dardanelle Dam, with an average depth at normal pool level of 9.1 ft. However, the navigation channel is generally deep due to degradation. Thalweg depths in 1972 and 1980 were a minimum of 17 ft below normal pool. The pool has good sinuosity, and crossings in the upper portion of the pool, except for the long crossing at N Miles 97-95, appear to have performed well in recent years with the additional contraction provided. Top of piling is above normal pool level

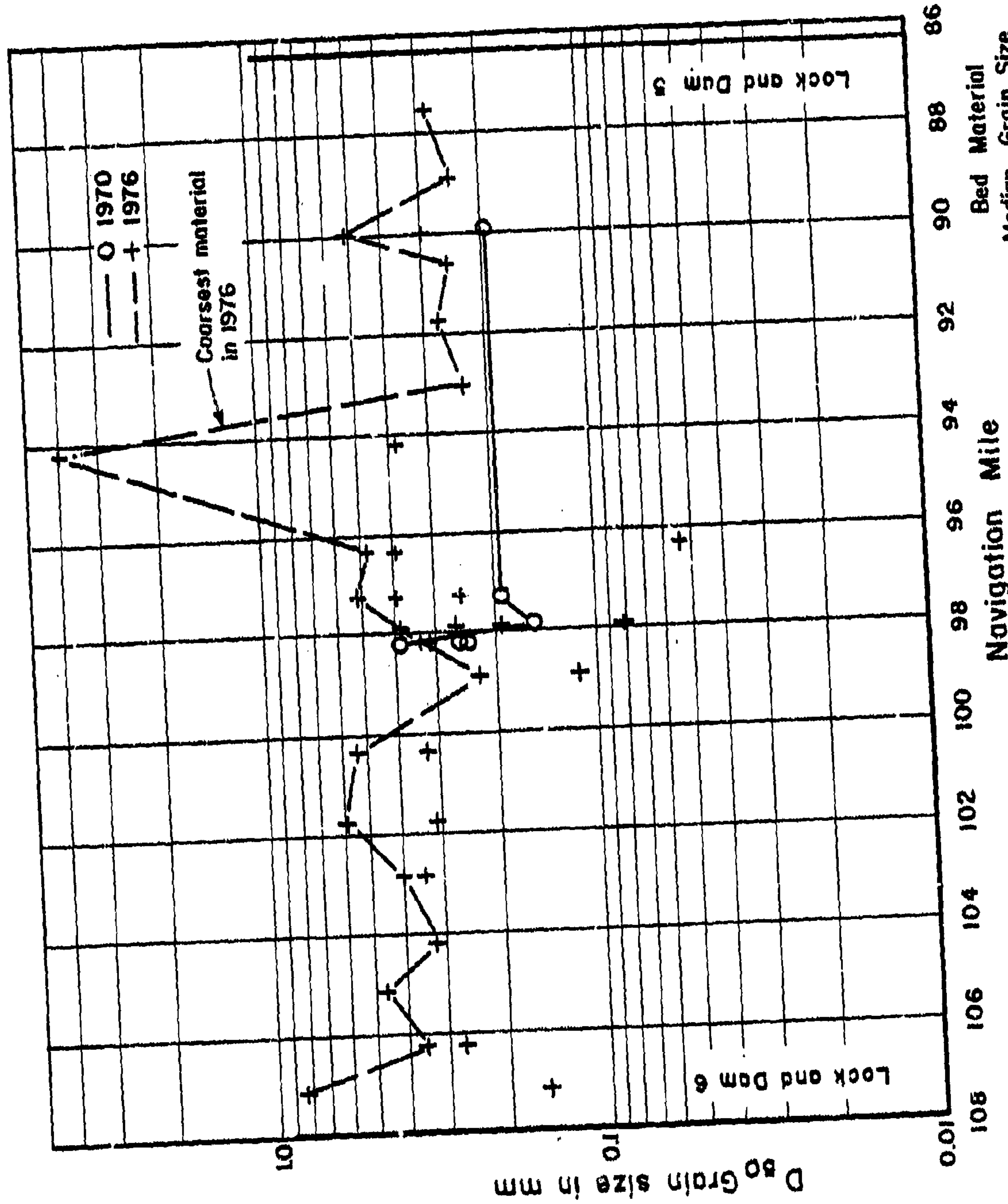

Lock and Dam 5
Lock and Dam 6
O 1970
+ 1976
Coarsest material in 1976
Bed Material
Median Grain Size
Pool 5
Navigation Mile
86
88
90
92
94
96
98
100
102
104
106
108
D50 Grain size in mm
1.0
0.1
0.01

Figure 2-13

throughout most of the pool, and depths in the lower portion of the pool are generally greater than 20 ft.

Storage capacity at normal pool level is 61,000 ac ft; this is almost twice the storage of Pool 6 which is about the same length. The pool is gate-controlled up to a discharge of 145,000 to 150,000 cfs; flows of this magnitude and higher are expected to prevail on the [illegible]ge about four percent of the time and appear to have occurred in 16 of the 22 years following closure of Dardanelle Dam, as discussed in Chapter 6.

2-7 POOL 4

Degradation - Data indicate little change in the thalweg profile through Pool 4 between 1960 and 1980; however, the 1971 and 1980 profiles appear to have an adverse slope in the seven-mile contracted reach downstream of Dam 5, Figure 2-6. The thalweg lowered several feet at the head of the pool between 1971 and 1980, but there was little aggradation at the downstream part of the pool. This may be related to Boyd Point cutoff, opened in 1962, that shortened the river by 5.75 miles. In 1969, 232,000 cu yd of material was dredged from the channel downstream of the cutoff at N Mile 70; however, the thalweg remained high in that area in 1971 and 1980, Figure 2-6. Available data do not indicate any additional contraction was required in Pool 4 after the project became operational.

Bed Material - Limited data available on bed material composition indicate median grain size was about 0.35 mm and was approximately constant

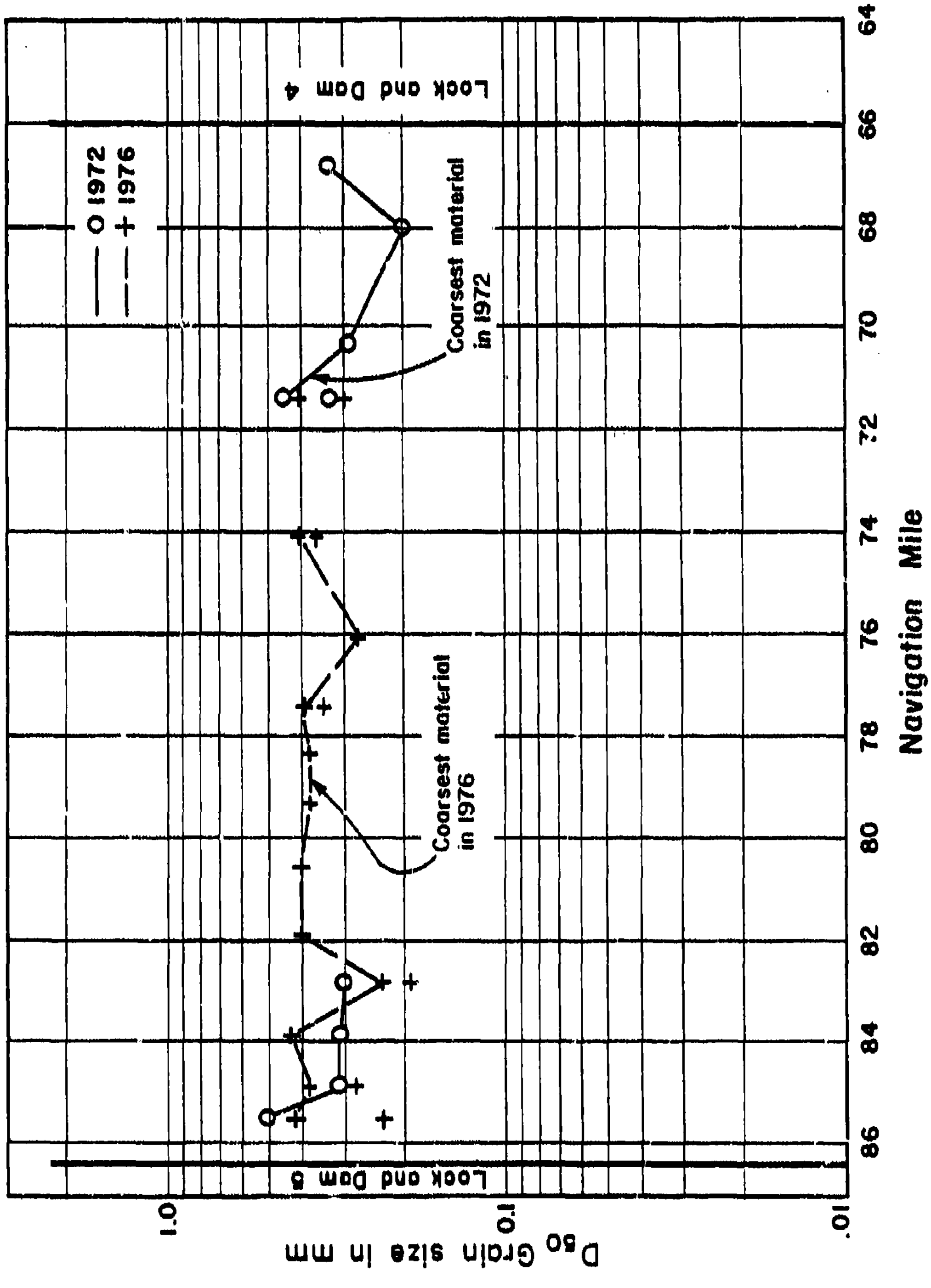
throughout the pool, Figure 2-14. Data do not indicate any coarsening of the bed between 1972 and 1976.

Water Surface Slope - Water surface drop through the reach was 20.5 ft for the 1957 flow of 450,000 cfs and 17.0 ft for the 1973 flow of 350,000 cfs. Water surface slopes for the 1957, 1973, and 1985 high flows were about the same, 0.78, 0.83, and 0.82 ft per mile, respectively.

Maintenance Dredging - About 215,000 cu yd was dredged immediately below Lock and Dam 5 from 1969 through 1975, and only 13,000 cu yd in the eight years following. About 243,000 cu yd was dredged downstream of Boyd Point cutoff, most of it in 1969; no dredging has been required in that area since 1972.

Discussion - Pool 4 is 20.3 miles long, with storage capacity of 70,000 ac ft at normal pool level. It is one of the deeper pools downstream of Dardanelle Dam with an average depth of 12.4 ft at normal pool. Storage of 70,000 ac ft at Pool 4 compares with storage of 86,000 ac ft at Pool 7 and 110,000 ac ft at Pool 2. Pool 4 is set higher with respect to the construction reference plane than Pool 5, Figure 6-1, but top of piling in control structures (at about 16 ft above the reference plane) would be at or above normal pool level except in the lower six miles of the pool.

Again, there is no indication of deposition in recent years. No maintenance dredging has been required in Pool 4 since 1975 except for 13,000 cu yd dredged in the lower approach to Lock 5 in 1983. No additional contraction works were needed. Pool 4 has been one of the three pools downstream of Dardanelle Dam requiring the least amount of maintenance



Bed Material
Median Grain Size
Pool 4

Figure 2-14

dredging in the 1969-1984 period. If the dredging downstream of Boyd Point cutoff in 1969 is disregarded (on the basis that it would not have been required if the channel had become stabilized prior to impoundment of the pool), dredging would have totaled only 179,000 cu yd or about the same as for Pool 6.

Pool 4 is gate-controlled up to a discharge of 145,000 to 150,000 cfs, and flows of this magnitude and higher are expected to prevail on the average about four percent of the time and appear to have occurred in 16 of the 22 years following closure of Dardanelle Dam, as discussed in Chapter 6.

2-8 POOL 3

Degradation - Data indicate little change in thalweg elevation throughout Pool 3 from 1960 to 1981 except that the rectified alignment has deepened the crossings at N Miles 57 and 55 in the downstream portion of the pool, Figure 2-7.

Bed Material -Information on bed material composition is available only for 1972. These data indicate the D_{50} of the bed material was about 0.3 mm and was approximately uniform throughout the pool at that time, Figure 2-15.

Water Surface Slope - The water surface slope for a flow of 350,000 cfs in 1973 of 0.65 ft per mile is somewhat flatter than the slope of 0.72 ft per mile in 1957 for a flow of 450,000 cfs but about the same as for the 1985 profile (0.67 ft per mile).

Maintenance Dredging - There has been almost annual maintenance dredging at N Miles 66-65 immediately downstream of Lock and Dam 4. A total

Bed Material
Median Grain Size
Pool 3

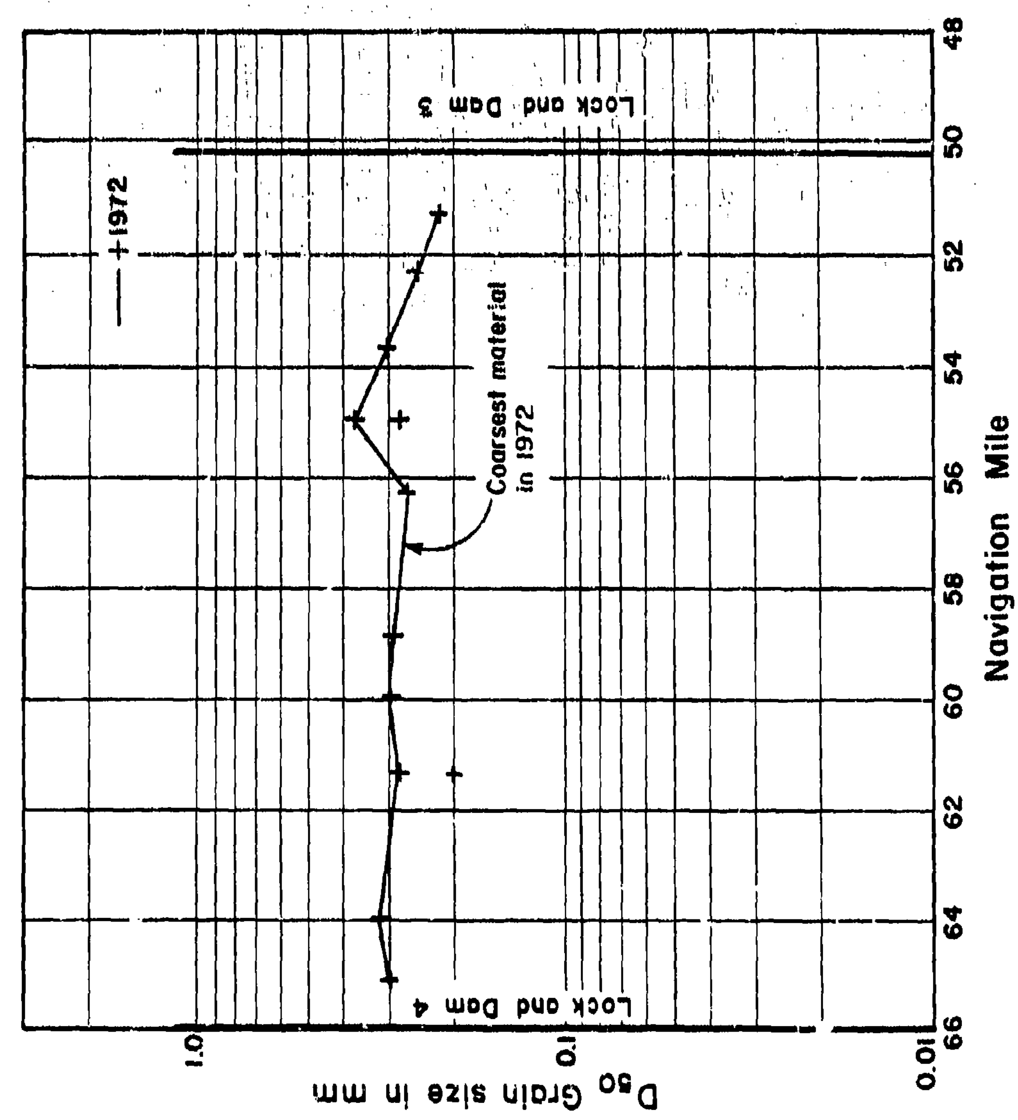


Figure 2-15

of about 360,000 cu yd of material has been removed from the area, 72 percent of it in the eight years 1969 through 1976, and 28 percent in the eight years 1977 through 1984. Although the amount of dredging has lessened with time, it has continued, indicating sediment is being supplied from the pool upstream. Discussion with Little Rock District personnel indicates that maintenance dredging begins when flow on the recession side of the hydrograph reaches 120,000 cfs (about the discharge at which gates can be operated to maintain the pool) without waiting for natural scour to occur as the flow continues to drop. Dredging in other areas of the pool has been minor: 50,300 cu yd in the crossing at N Mile 64 in 1973 and 36,000 cu yd at N Mile 51 just upstream of Lock and Dam 3 in 1969.

Discussion - Pool 3 is the shortest pool downstream of Dardanelle, with a length of 15.8 miles and an average depth of 12.4 ft. At the head end, the pool is set 8.5 ft above CRP, making it the highest pool with respect to CRP in the system, Figure 6-1. (The head of Pools 6 and 4 are 3.5 and 4 ft, respectively, above CRP.)

Storage at normal pool is 46,000 ac ft. Only Pool 8 has lesser storage (32,000 ac ft) at normal pool. Pool 3 is gate-controlled up to a flow of about 125,000, a flow equaled or exceeded on the average about seven percent of the time, and open-river conditions appear to have occurred in 18 of the 22 years following closure of Dardanelle Dam, as discussed in Chapter 6.

Normal pool is set very high with respect to CRP, and flows greater than about 60,000 cfs appear to be over the top of stone in the control

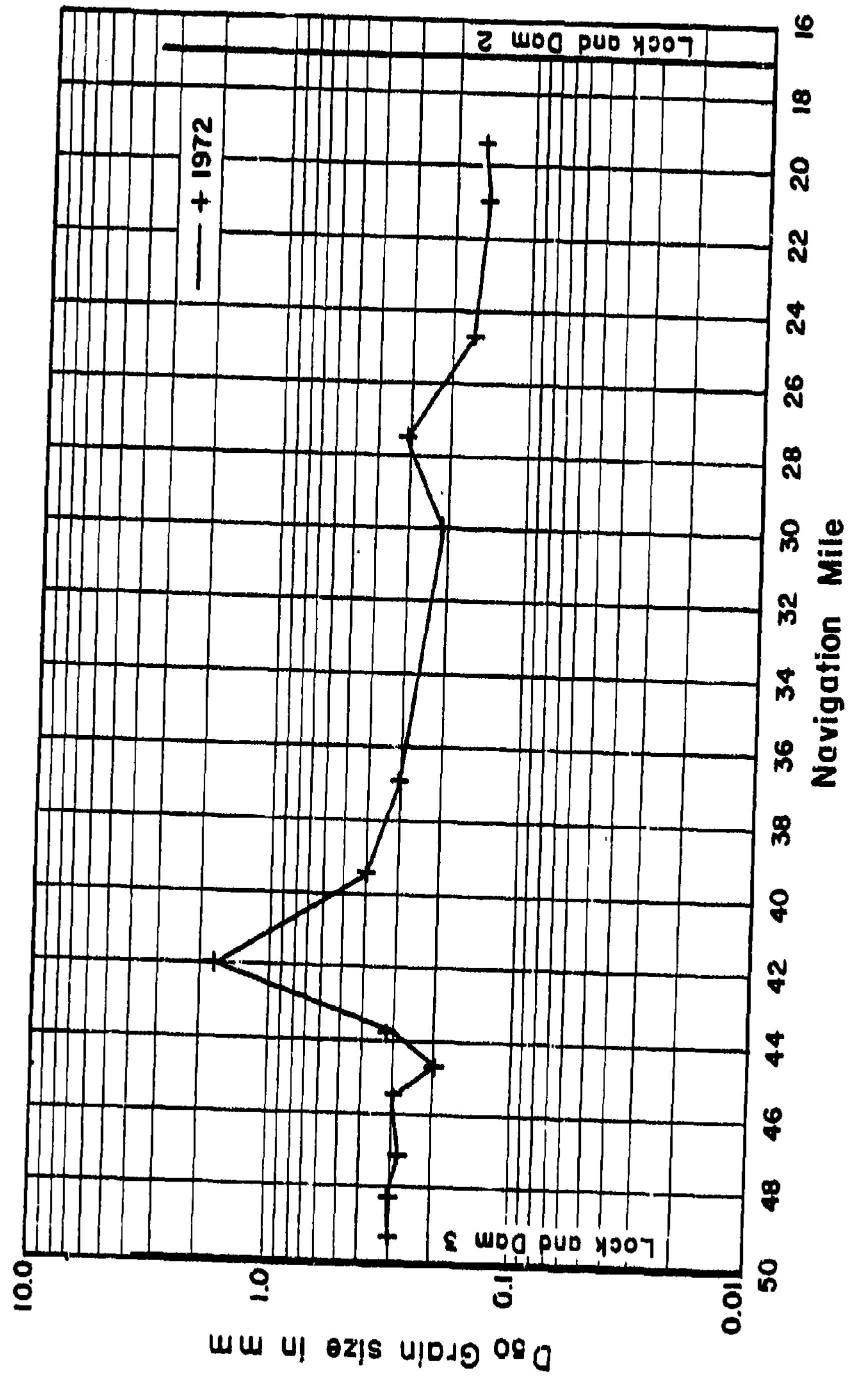
structures; thus, the structures are not effective in constricting the channel at moderate to high flows. The pool has little sinuosity in the nine-mile reach from N Mile 64 to 55, and the crossing at N Miles 55-52 is very long; however, no maintenance dredging or additional contraction have been required in those areas.

2-9 POOL 2

Degradation - There appears to have been little change in thalweg elevation throughout Pool 2 except in the five miles immediately downstream of Dam 3, where the 1980 thalweg was 10 to 15 ft below 1961 conditions, and in the lower 10 miles of the pool from N Mile 27 to Dam 2, Figure 2-8. In the latter reach, the thalweg had lowered in the order of 5 to 15 ft between the early 1960's and 1972 due to contraction and rectification work. However, there has been significant deposition (in the order of 10 to 30 ft) in the lower end of the pool, particularly in the reach between N Miles 26 and 20. Construction of Little Bayou Meto cutoff in 1962 in the head of the pool shortened the reach by 5.6 miles.

Bed Material - Information on bed material composition is available only for 1972. These data indicate the D_{50} of the bed material decreased slightly through the pool at that time, ranging from about 0.3 mm at the head end to about 0.2 near Dam 2, Figure 2-16.

Water Surface Slope - The overall water surface slope through the reach for the June 1957 flow of 450,000 cfs was 0.62 ft per mile, or less steep



Bed Material
Median Grain Size
Pool 2

Figure 2-16

than the slope of 0.66 ft per mile for the April 1973 flow of 350,000 cfs and the slope of the 1985 profile (0.70 ft per mile).

Maintenance Dredging - Dredging in Pool 2 totaled approximately 48 percent of all maintenance dredging downstream of Dardanelle Dam in the 1969-1984 period. In the six miles at the head of Pool 2 dredging has been performed annually (except in the low-flow year of 1981), and in some cases as many as eight times during a single year, in the period 1969-April 1985. Total volume dredged in that reach was about 7.6 million cu yds. There has also been maintenance dredging at a number of other locations throughout the pool, including about one million cu yds in recent years at the following locations:

- 1985, N Mile 43, downstream end of a long crossing - 29,000 cu yd
- 1982, N Mile 33, downstream end of a long crossing - 76,800 cu yd
- 1983-84, N Mile 32, bend - 745,500 cu yd
- 1983, N Mile 28, crossing - 57,600 cu yd
- 1982, N Mile 24, flat bend of little curvature - 98,100 cu yd
- 1985, N Mile 19, bend - 29,200 cu yd

Discussion - Pool 2 has a length of 33.2 miles and is the longest pool downstream of Dardanelle Dam. It has a storage volume at normal pool of 110,000 ac ft and is the largest pool downstream of Dardanelle. In these characteristics it resembles the Ozark pool upstream of Dardanelle that has

a length of 36 miles and storage volume of 148,000 ac ft at normal pool. Downstream of Dardanelle, only Pool 7 with a length of 30.5 miles and storage volume of 86,000 ac ft approaches conditions in Pool 2, Table 2-1.

The five-mile reach immediately below Lock and Dam 3 was relatively straight under preproject conditions and was expected to be a difficult reach in which to maintain adequate navigation depth during project design. Accordingly, as much sinuosity and contraction were designed into the reach as appeared feasible considering costs and impacts on flood heights. Throughout the remainder of the pool, the channel has generally good sinuosity and crossing are not abnormally long.

At the head of the pool, normal pool level is 1.5 ft below CRP. Because of the length of the pool, CRP is 26 ft below normal pool at Dam 2, and the top of piling in the control structures is generally below pool level in the lower 10 miles of the pool. Open-river conditions begin at an elevation that is high with respect to top of control structures (at a flow of about 280,000 cfs) so that rock in the control structures is submerged about 15 ft when open-river flow begins. Accordingly, structures in Pool 2 are not completely effective in constricting flow and confining it to the main channel.

Flow through Pool 2 is gate controlled for discharges up to about 280,000 cfs (a flow expected to occur less than one percent of the time) for a low Mississippi River downstream. While design studies indicated a flow of that magnitude would have a recurrence interval on the average of about

once in four years, a discharge of 270,000 cfs or higher at Little Rock occurred in only 3 of the 22 years following closure of Dardanelle Dam, indicating a return period of about seven years. With the Mississippi River at high stage, backwater effects extend upstream on the Arkansas River into Pool 2. The fact that the pool is controlled for discharges up to about 280,000 cfs indicates that open-river conditions rarely exist through Pool 2 and that Pool 2 acts as a storage reservoir and a sediment trap. Under these conditions deposition of coarser material is to be expected at the head of the pool, with finer material deposited in the lower portion of the pool. [It would be very helpful to know the size distribution of bed material throughout Pool 2 under present (1986) conditions in hypothesizing on sediment transport through the pool.] As the lower end of the pool aggrades, the water surface at the head end of the pool rises for a given discharge, increasing the depth of flow and decreasing the velocity and transport capacity, and thus causing aggradation at the head of the pool.

It appears that upstream pools are more or less in a state of equilibrium with regard to sediment transport, but that detention time in Pool 2 is sufficiently long to cause the pool to trap significant volumes of sediment annually that have passed through the upstream navigation pools.

CHAPTER 3 - REPRESENTATIVE DEPOSITION PROBLEMS

3-1 GENERAL

Discussion of deposition problem areas in Chapter 2 indicates that, except for Pool 9 and a few other minor isolated exceptions, all maintenance dredging required has been in the heads of the low-lift pools in the reach downstream from Dardanelle Dam, and that except for Pool 2, the bulk of the maintenance dredging (83 percent) was performed in the early years of project operation, prior to 1976. There was significant initial dredging as a part of project construction at the heads of Pool 9 through 2, as discussed in Chapters 2 and 4, (including 17 million cu yds in Pool 9), that enlarged the natural cross section in those reaches. Initial excess channel enlargement perhaps contributed to deposition for several years as the rectified and canalized channel approached equilibrium.

Typical problem areas in the upper portions of the pools were generally relatively long straight reaches, reaches of flat curvature, and long crossings. The channel was constricted in some local areas in Pools 9 through 5 after the project went into operation to minimize deposition and to improve the reliability of adequate navigation depth.

3-2 LOCAL PROBLEM AREAS

Local areas of deposition requiring maintenance dredging, with the exception of areas immediately downstream of the locks and dams, are described below.

Pool 9, N Mile 195-188 - A seven-mile essentially straight reach with little sinuosity, a poorly defined crossing at miles 193-192, and a second crossing at mile 190 narrowed to achieve better direction of flow after the project became operational. Original design width ranged from 1,300 to 1,000 ft and was narrowed to 900 to 800 ft between miles 190 and 187. Overall layout of the rectified channel was constrained by rock escarpments on the left and right banks. There has been no maintenance dredging in this reach since 1979, Figure 2-1, due largely to effects of Dardanelle Dam upstream. It is unlikely a similar layout at a site farther downstream would have functioned as well.

Pool 8, N Mile 173-170 - A long, poorly defined crossing that was contracted from a design width of 1,200 to 600 ft in 1975. No dredging has been required since the reach was narrowed, Figure 2-2.

Pool 7, N Mile 153-149.5 - A long, poorly defined crossing downstream of a large radius bend. Downstream from mile 151, channel was narrowed from 1,100 to 600 ft in 1970, Figure 2-3. Despite narrowing, dredging has continued to be required; however, deposition appears to be more a problem of pool characteristics than of the stabilization and rectification design, except possibly structure height in relation to initiation of open-river conditions, are discussed in Chapter 6. The effects of pool characteristics, including the frequency of occurrence of open-river flow conditions on deposition in the navigation pools, is discussed further in Chapter 6.

Pool 7, N Mile 147-145 - Downstream portion of a moderate radius bend narrowed from 1,000 to 700 ft in 1971. Maintenance dredging has continued to be required in the 1980's, but deposition appears to be more of a problem of pool characteristics such as length, depth, and storage than of stabilization and rectification design.

Pool 7, N Mile 144-141 - A three-mile long crossing at about mid-pool. Channel was narrowed from miles 142.5 to 141.5 by L-heads and vane dikes from 1,400 to 700 ft in 1972, and no dredging has been required since.

Pool 6, No Problem Areas - Reach from N mile 118.8 to 116 was narrowed from 1,200 to 800 ft in 1968 to guide navigation through the series of bridges at Little Rock, not to increase depth, Figure 2-4.

Pool 5, N Mile 105-103.5 - A long crossing near the head of the pool, narrowed from 1,100 to 700 ft with vanes. No dredging has been required, Figure 2-5.

Pool 5, N Mile 100-98.5 - A 1.5-mile crossing narrowed at downstream end from 1,400 to 1,000 ft in 1971. No dredging required after 1972.

Pool 5, N Mile 97-95 - A two-mile long crossing at mid-point of pool. No dredging required since 1979.

Pool 4, No Problem Areas - Figure 2-6.

Pool 3, No Problem Areas - Figure 2-7.

Pool 2, N Mile 50-44 - This six-mile reach has good sinuosity; CRP is high with respect to normal pool elevation at head of pool; but excessive maintenance dredging has been required from 1969 to present, Figure 2-8.

Deposition problems in the reach appear to be related to pool characteristics, not to design of stabilization and recitification work except possibly structure height in relation to initiation of open-river conditions, as discussed in Chapter 6.

CHAPTER 4 - MAINTENANCE DREDGING

4-1 GENERAL

Sediment deposition and maintenance dredging in each pool are discussed in Chapter 2. This chapter is limited to a more general discussion of the overall dredging problem downstream from Dardanelle Dam.

The authorized navigation channel on the Arkansas River is 250 ft wide and nine feet deep. Initial dredging at the heads of some pools, Table 4-1, was included as a part of project construction to hasten development of an equilibrium degraded channel that would provide navigable depth with a minimum of maintenance dredging and meet the scheduled dates for initiating navigation to Little Rock in 1968, to Fort Smith in 1969, and to Catoosa-Tulsa in 1970. Initial dredging limits often were considerably more than that needed for navigation. For example, in the upper end of Pools 9, 13 and 14, the channel was initially dredged up to 400 ft wide and 16 ft deep.

Typical maintenance dredging is to a depth of 12 ft, including 3 ft of overdepth dredging for "advance maintenance" to allow a time period for sediment buildup before the 9-ft authorized depth is no longer available and maintenance dredging must be repeated.

In the design phase of the project, it was recognized that the upper ends of both Pools 2 and 7 would be problem shoaling reaches. Consequently, movable bed physical model studies were initiated at Waterways Experiment Station in the early 1960's to develop effective bank stabilization and

rectification systems for those reaches. However, even the structure layouts developed in the models did not eliminate all channel maintenance dredging requirements.

Prior analyses of maintenance dredging have focused on average values of maintenance dredging with respect to pools and time, the effectiveness of stabilization and contraction works at the heads of the pools, and the identification of problem pools (1). While the impact of pool characteristics on maintenance dredging requirements has been recognized, this aspect does not appear to have been addressed in detail. Current studies indicate that pool length and depth and frequency of open-channel flow conditions may be the primary causes of atypical deposition and maintenance dredging requirements in Pools 2 and 7, as discussed further in Chapters 2 and 6.

4-2 INITIAL DREDGING

In design studies for the Arkansas River project, estimates were made of progressive degradation and armoring of the bed downstream from Dardanelle Dam. Spillway sills of the low-lift navigation dams downstream were set near the computed future degraded bed levels, and in some cases those elevations were significantly lower than current bed elevation at time of construction. Accordingly, to provide adequate navigable depth in the early years of operation in the downstream lock approaches, initial dredging was

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1. Maintenance Dredging on the Arkansas River, T. Schmidgall, in Sedimentation, H. W. Shen, ed., 1976.

required as a part of project constuction for a distance of several miles downstream from the low-lift structures. Initial volumes dredged in each pool as a part of project construction are tabulated in Table 4-1. Initial dredging removed essentially a wedge of material from the bed downstream of the structures, with the apex of the wedge in the order of three to five miles below the dams in most pools.

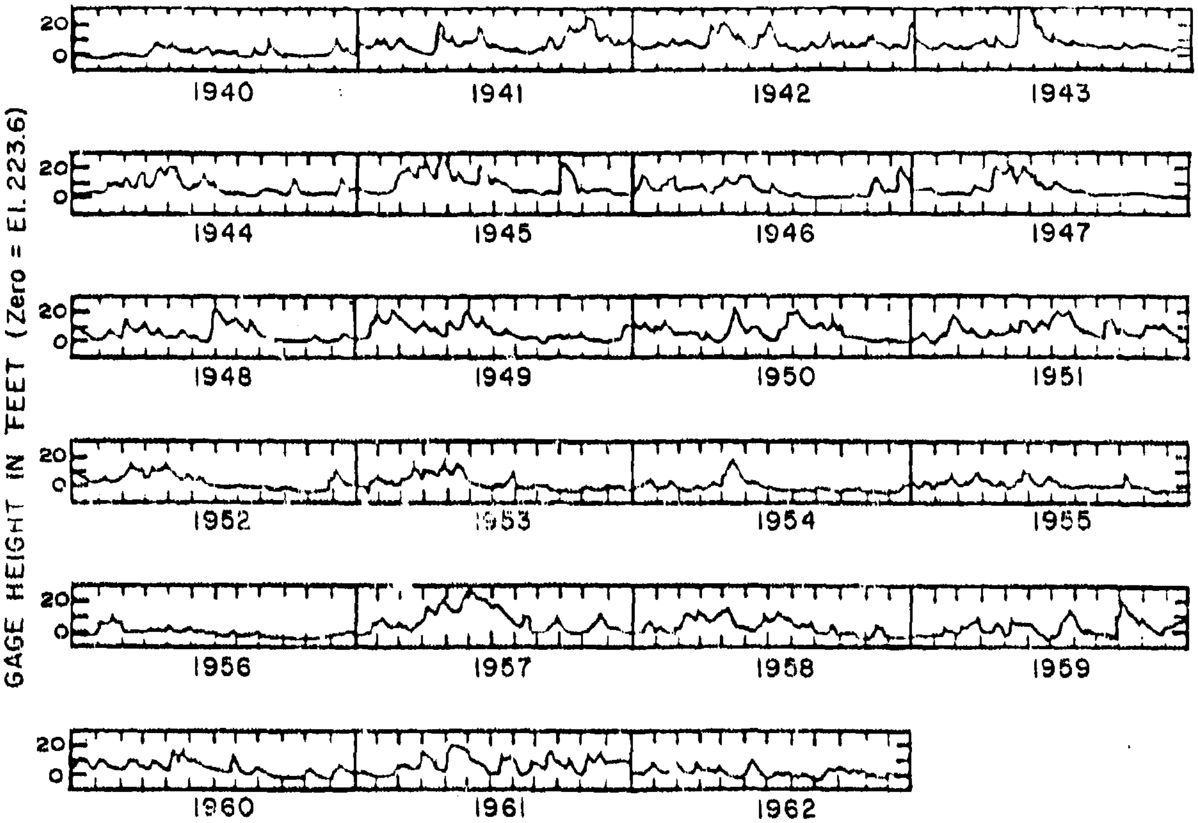
Table 4-1

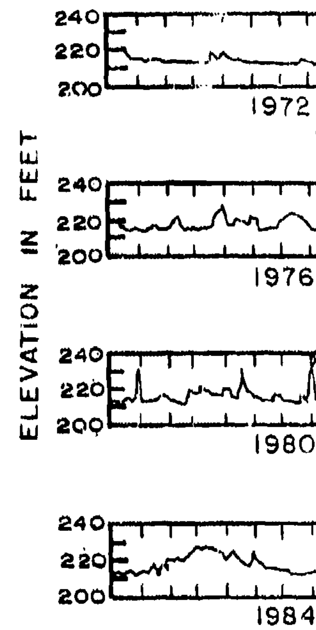
Initial Dredging Volumes

Pool	Volume (1,000 cu yd)
9	17,000
8	4,900
7	8,300
6	2,500
5	2,600
4	100
3	100
2	3,200

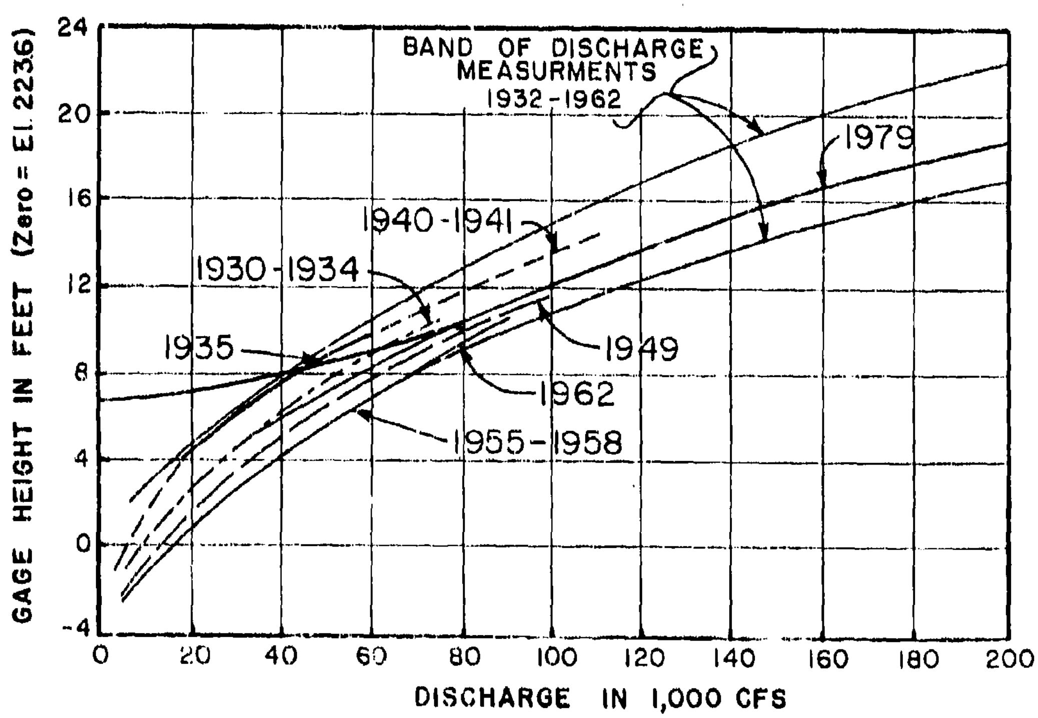
4-3 RELATIONSHIP OF STREAMFLOW AND MAINTENANCE DREDGING

As discussed in Chapter 1, streamflow in the lower Arkansas River, for example at Little Rock, was highly variable under preproject conditions. Flow has continued to be variable under project conditions, but with less extreme high and low flows, as indicated by stage data at Little Rock and Lock and Dam 6, Figure 4-1, and by discharge data in Table 5-2.

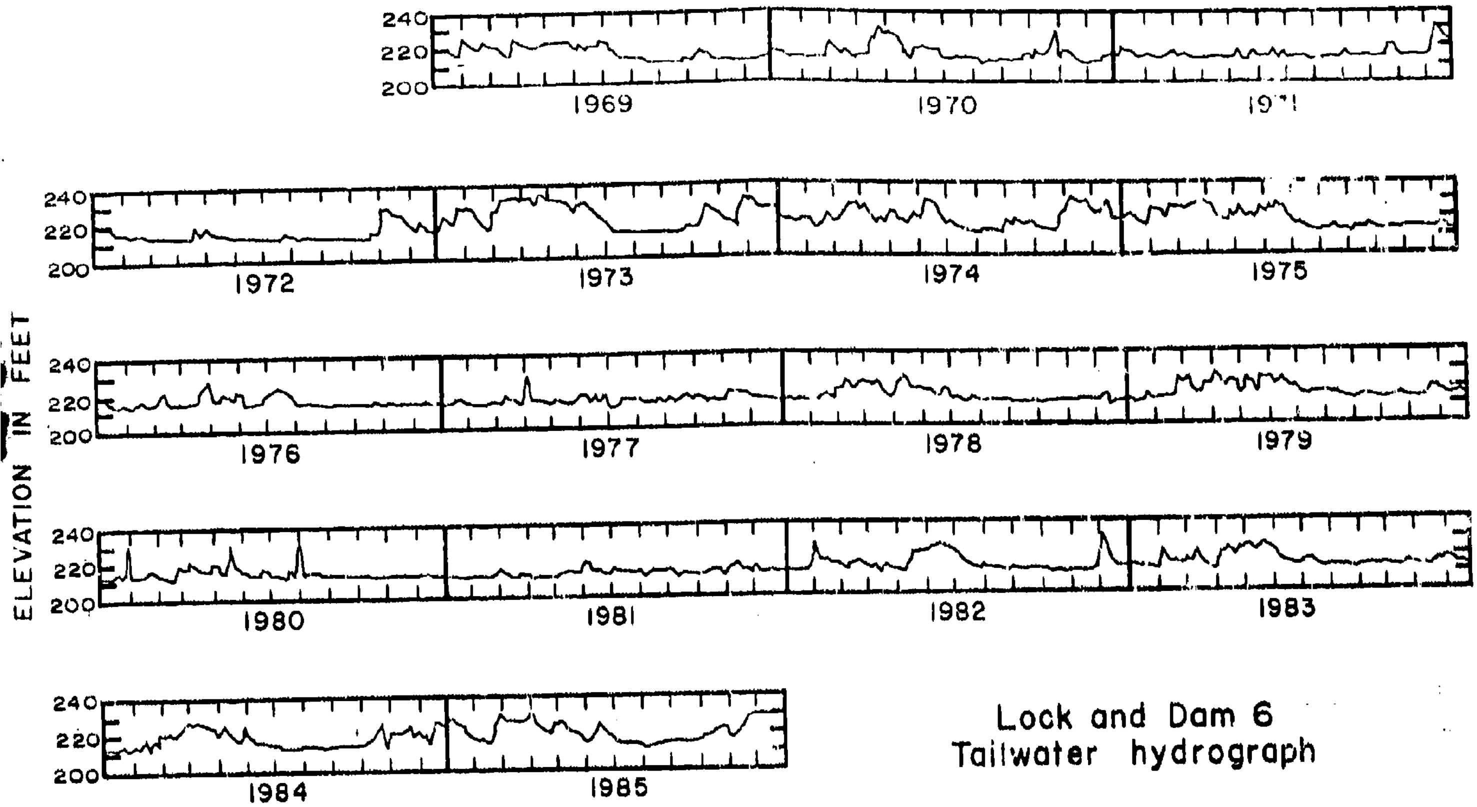




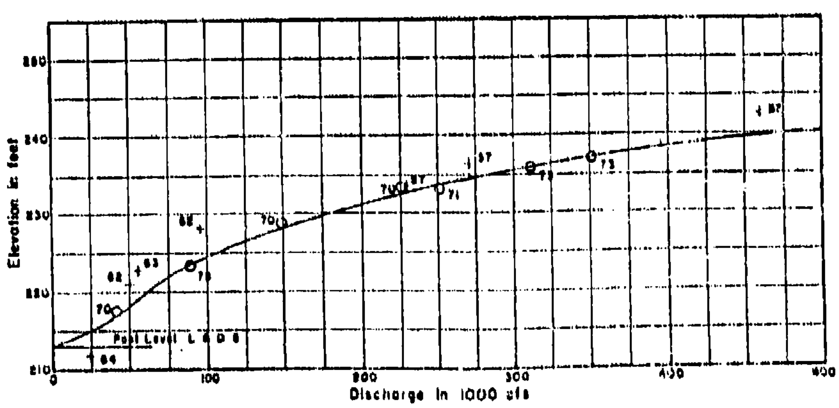
Little Rock Stage Hydrograph



Discharge Rating Curves



Lock and Dam 6
Tailwater hydrograph



Lock and Dam 6
Tailwater Rating Curve

Arkansas River
Little Rock, Arkansas
Stage Hydrographs and
Discharge Rating Curves

2

Figure 4-1

Schmidgall examined maintenance dredging requirements as related to streamflow (1). He concluded that the amount of dredging required in most pools is related to volume of flow. Annual maintenance dredging in the State of Arkansas (the lower reach of Pool 13 through Pool 2) is related to annual streamflow at Van Buren (in Pool 13) in Figure 4-2, based on Schmidgall's data (2). Maintenance dredging by years in Pools 9 through 2 and annual runoff at Lock and Dam 13 (Van Buren) are shown in Table 4-2.

In the 46-year period 1928-1975, average annual discharge at Van Buren was 22 million ac-ft and 29 million ac-ft at Little Rock. Data in Figure 4-2, Table 4-2, and Table 5-2 indicate that:

1. In early years of project operation (1970-1972) discharge was near average.
2. The next two years (1973-1974) were years of abnormally high stream flow, requiring large volumes of maintenance dredging, and in 1975 flow as somewhat above average.
3. The years 1976-1979 were years of average flow.
4. The years 1980-1981 were years of abnormally low flow, requiring essentially no maintenance dredging.
5. Flows in 1982-1984 were about average.

As Schmidgall noted (2), a year of median flow with several flood peaks can require more maintenance dredging than a high volume year with only one

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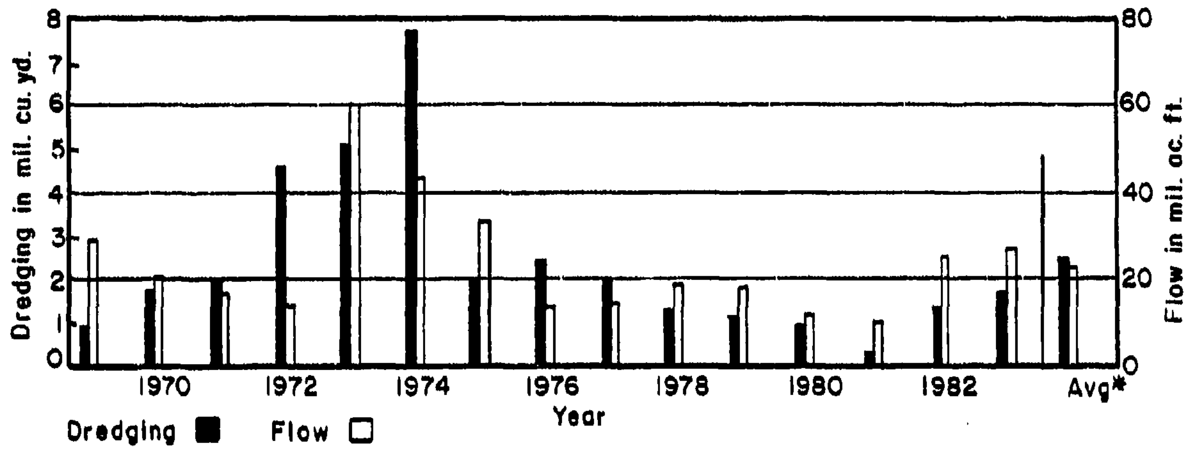
2. D.F. McClellan-Kerr Arkansas River Navigation System, An Engineering Evaluation, Schmidgall, 1 June 1981.



* Flow, 52-yr average at Van Buren, Arkansas
Dredging, 14-yr average in Arkansas

Arkansas River
Annual Maintenance Dredging
in Arkansas and Annual Flow

Figure 4-2

Table 4-2

ARKANSAS RIVER

Maintenance Dredging Pools 9 through 2 (1000 cu yd)

Pool	Year																	
	69	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	Total	% Total
9		0	50.3	162.2	219.9	393.7	0	48.6	0	39.1	108.4	0	0	81.2	0	47.5	1,150.7	6.2
8	--	262.4	59.0	612.0	774.6	749.2	175.8	102.3	38.8	0	0	0	0	0	0	0	2,774.3	15.1
7		187.2	153.7	432.0	798.3	646.3	324.5	232.0	42.4	0	19.1	0	0	151.7	161.1	30.0	3,178.7	17.3
6	--	24.2	61.2	35.7	47.3	0	0	0	0	0	0	0	0	0	0	0	168.2	0.9
5	141.5	181.7	306.6	70.2	101.0	111.5	16.5	24.0	106.6	32.2	138.5	0	0	118.4	0	0	1,348.7	7.3
4	301.7	34.6	5.3	25.3	37.1	37.4	26.5	0	0	0	0	0	0	0	13.0	0	481.0	2.6
3	75.6	54.5	--	11.1	120.5	16.8	49.0	19.1	0	0	9.0	0	0	64.3	21.0	6.5	447.4	2.4
2	268.8	322.1	226.5	773.[illegible]	946.7	1493.7	496.0	733.8	557.3	448.3	376.5	22.1	0	598.5	814.5	774.1	8,852.2	48.1
Total	787.6	1066.7	862.6	2122.1	3045.4	3449.1	1088.3	1159.8	745.1	519.6	651.5	22.1	0	1014.1	1009.6	858.1	18,401.2	100.0
% Total	4.3	5.8	4.7	11.5	16.5	18.7	5.9	6.3	4.0	2.8	3.5	0.1	0	5.5	5.5	4.7		100.0
Cumulative	788	1854	2717	4839	7884	11,333	12,422	13,582	14,326	14,846	15,498	15,520	15520	16,534	17,544	18,402		

Maintenance Dredging Pools 9 through 3 (1000 cu yd)

	69	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	Total	% Total
Total	518.8	744.6	636.1	1348.5	2098.7	1955.4	592.3	426.0	187.8	71.3	275.0	0	0	415.6	195.1	84.0	9,549.0	
% Total	5.4	7.8	6.7	14.1	22.0	20.5	6.2	4.5	2.0	0.7	2.9	--	--	4.4	2.0	0.9		100.0
Cumulative	518.8	1263	1900	3248	5347	7302	7894	8320	8508	8580	8854	8854	8854	9270	9465	9549		

Annual Runoff at Lock & Dam 13 (million ac ft)

	69	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84
Annual	29	21	17	14	61	44	34	14	15	17	18	11.6	9.7	24.8	25.7	25.8
Cumulative	29	50	67	81	142	186	220	234	249	266	284	295.6	305.3	330.1	355.8	381.6

or two peaks and that if high flows occur late in the year, the required dredging is usually accomplished and recorded in the following calendar year. These factors obscure a clear-cut relationship between annual discharge and required maintenance dredging in a given year.

4-4 MAINTENANCE DREDGING POLICY

Discussion with Little Rock District personnel indicates that the District operates under a policy of providing authorized navigable depth 100 percent of the time to the extent feasible. To execute this policy, dredging begins on the hydrograph recession at flows in the order of 120,000 to 70,000 cfs (flows that carry a significant sediment load with depths considerably in excess of authorized depth) to minimize potential interruption of navigation. As a result, deposition often continues as flow decreases, and repeated dredging is sometimes required on one hydrograph recession, particularly in reaches in the head of Pool 2.

4-5 MODIFICATION OF OPERATING CRITERIA FOR UPSTREAM STORAGE RESERVOIRS

Major floods on the lower Arkansas River have their source in the upper basin in Oklahoma. Reservoirs in Oklahoma were originally operated for flood control by making releases to limit discharge at the Oklahoma-Arkansas state line to from 105,000 to 150,000 cfs (depending on the degree of encroachment in the flood control space in the reservoirs) until flood control space was emptied; releases were then decreased abruptly. That original operating procedure resulted in a sudden loss of sediment-transport

capacity and extensive shoaling in crossings in the lower river. Schmidgall (1) discusses modification of the operating plan to control flow to 40,000 cfs at the state line for about two weeks on the recession side of flood hydrographs until flood storage is completely evacuated to lessen shoaling in the lower river and allow time to identify shoaled reaches and mobilize dredging equipment. Although benefits are not easily identified, flood recession release schedules from upstream reservoirs are still (1986) being studied in an effort to develop a schedule that will minimize shoaling tendencies in the Arkansas River navigation channel.

4-6 EFFECT OF UPSTREAM POOL

In project design studies, pools with high lift and significant storage capacity, such as Ozark and Dardanelle, were expected to trap considerable sediment and to release relatively clear water that would regain its normal sediment load from the channel of the next pool (or pools) downstream. Such sediment pick up from the bed and banks has occurred in Pool 9 below Dardanelle Dam, and in recent years, in Pool 8, apparently as the effects of Dardanelle have extended downstream. Dredging requirements in Pools 9 and 8 have been negligible in recent years, Table 4-2. Available data indicate that annual sediment discharge at Little Rock is about three times the load carried by releases from Dardanelle Dam, as discussed in Section 5-10.

In design studies it was not considered that Pools 7 and 2 might impound sufficient storage and reach open-river conditions at such relatively infrequent intervals as to cause those two pools to act as

sediment traps; however, it appears this is occurring, as discussed in Section 6-4.

Effect of Pool 8 on Pool 7 Deposition - As discussed in Sections 2-3 and 6-4, Pool 8 is one of the shorter pools downstream of Dardanelle, has the smallest storage capacity, and is operated under open-river conditions almost annually, more frequently than any other pool. In contrast, open-river flow conditions through Pool 7 occur less frequently, having a return period of about 2.4 years. There has been no maintenance dredging in Pool 8 since 1977, indicating that sediment supply and transport through the pool have been in equilibrium in recent years. However, the volume of material transported through Pool 8 apparently generally exceeds transport capacity in the upper ten miles of Pool 7 where 3.14 million cu yds of deposited material were dredged from 1970 through 1984, 90 percent of it in the 1970's.

Effect of Pool 7 on Pool 6 Deposition -As discussed in Section 2-4, significant sediment deposits (about 3.14 million cu yds) have been dredged from the upper ten miles of Pool 7. Since that dredged material was generally disposed of outside the main channel (between dikes and landward of revetments) and vegetative growth has tended to stabilize the disposal areas, removal of that material from transport through Pool 7 may be a major contributing factor to the extremely small volume of sediment deposited in and dredged from Pool 6. It should be noted that Pool 6 is one of the two shortest pools in the system below Dardanelle, is one of the three smallest

in terms of storage, and appears to have operated with open-river conditions in 14 of the 22 years following closure of Dardanelle Dam (return period about 1.6 years), Table 6-1.

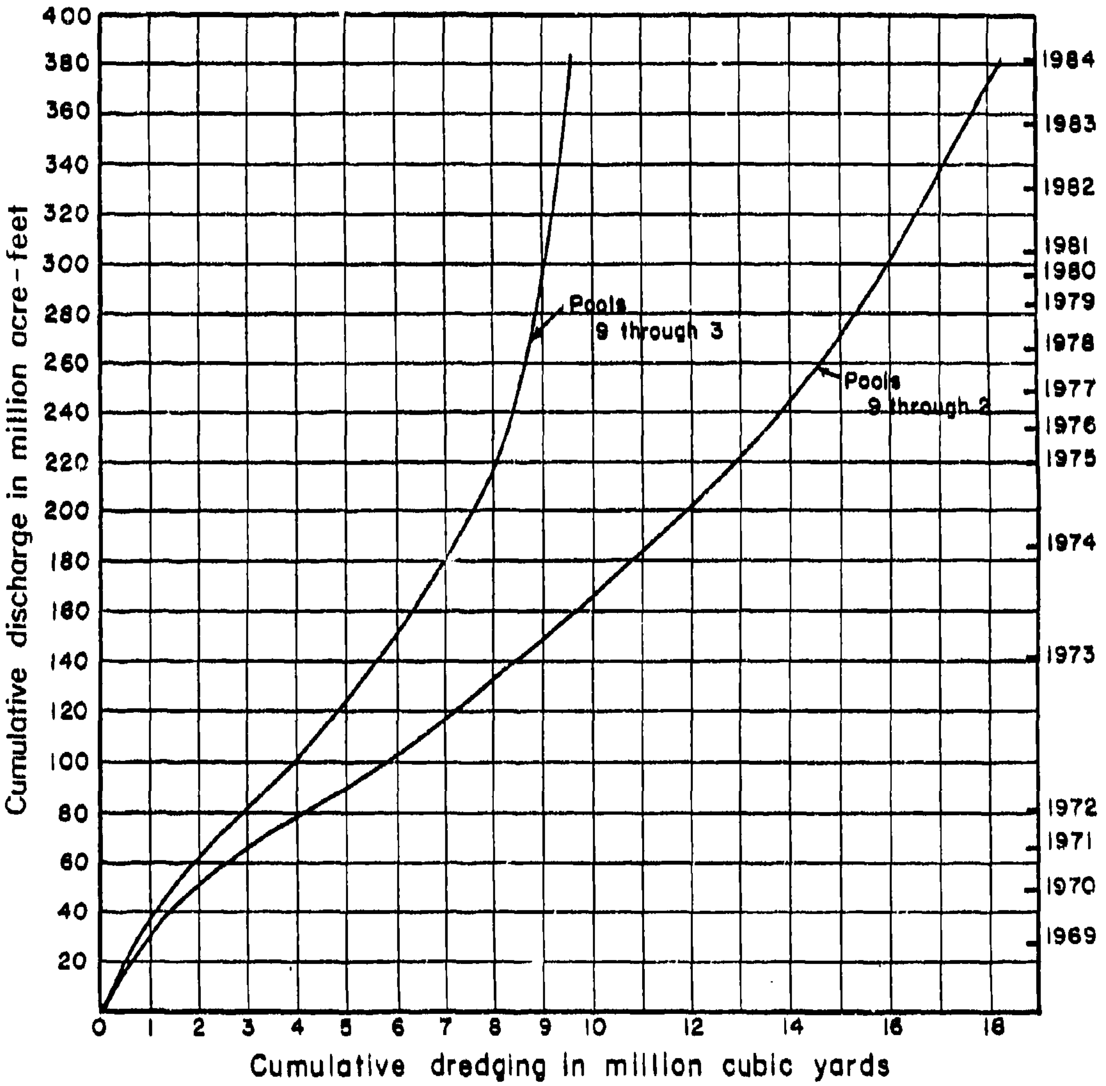
Effect of Pool 3 on Pool 2 Deposition - As discussed in Section 2-8, Pool 3 is the shortest pool downstream of Dardanelle Dam, has the second smallest storage capacity, and open-river conditions prevail with relative frequency, appearing to have occurred in 18 of the 22 years following closure of Dardanelle Dam, Table 6-1. Thus, it can be expected that sediment transport through the pool would be good and that the lesser transport capacity and longer retention time in Pool 2 downstream would result in deposition problems at the head of Pool 2. Such conditions have occurred. Pool 2 is deeper and is set much higher with respect to CRP than Pool 3. Also, Pool 2 appears to have operated at open-river conditions in only three of the 22 years following closure of Dardanelle Dam. Total material dredged from Pool 3 in the 1969-1984 period totaled was about 447,000 cu yds, but 8,852,000 cu yds was dredged from Pool 2 in the same period.

4-7 CURRENT STUDIES OF DREDGING IN POOLS 9 THROUGH 2

As discussed in Chapter 2, Pool 2 is clearly the pool experiencing the most severe deposition problems and requiring the most maintenance dredging (48.1 percent of all dredging downstream of Dardanelle Dam in the 1969-1984 period). Pools 7 and 8, with 17.3 and 15.1 percent of the maintenance dredging, rank second and third, respectively, in terms of severity of

deposition problems in the 1969-1984 period, but deposition in Pool 8 has not been a problem since 1977, apparently as a result of the effects of Dardanelle extending downstream with time.

Cumulative dredging volume since the project became operational in 1969 is shown in Figure 4-3 for two reaches: Pools 9 through 3 and Pools 9 through 2. Data in the figure indicate that if one disregards dredging in Pool 2 (on the basis that in view of its size, it is atypical of pools downstream of Dardanelle, as discussed in Section 2-9) annual dredging has decreased significantly with time over the period of study and has been at a relatively constant and negligible rate of 780 cu yds per 100,000 ac ft of flow (or 150,000 cu yds per year) for the period 1978-1984. However, if data for total dredging in Pools 9 through 2 are examined, it becomes clear that Pool 2 deposition problems are of a different order of magnitude (and probably of different origin) than those in Pools 9 through 3. Considering Pools 9 through 2, annual dredging has decreased with time, but at a far lesser rate than for Pools 9 through 3, and has remained at the relatively constant rate of 3,000 cu yds per 100,000 ac ft of flow (or 580,000 cu yds per year) for the period 1978-1984.



Arkansas River, Arkansas
Dredging as a function of discharge at Van Buren

Figure 4-3

CHAPTER 5 - SLOPES AND SEDIMENT TRANSPORT

5-1 GENERAL

Water surface slopes in Pools 9 through 2 and losses through the dams are summarized in Table 5-1. Water surface slopes for post-project conditions are average longitudinal slopes through each pool for discharges of sufficient magnitude that spillway gates were fully open and river flow was essentially at open-river conditions. Random scatter in the data probably is due to water surface elevations taken on rising or falling stages at different times and possibly due to water surface elevations affected by dynamic variations. Random scatter is ignored in the following discussion.

The preimpoundment (and essentially preproject) slopes of 1957 ranged from about 0.92 ft per mile in Pool 9 below Dardanelle Dam to about 0.39 ft per mile in Pool 2 above the Mississippi River, a general flattening of slope in the downstream direction typical of alluvial rivers. Slopes in individual pools vary from this general decrease somewhat for unknown reasons. However, local variations in most pools are not large and can be related to various factors. For the purpose of this study, it is the changes resulting from the project that are of special interest, not local variations.

Unfortunately, measurements of sediment transport available for this study were not extensive enough to trace change in sediment concentration with time and with confidence. Measured concentration-discharge values are

plotted in Figures 5-1 through 5-8. There is considerable scatter in the data (probably both real and random error) and no systematic change except that the few early values show larger concentrations than later ones.

5-2 POOL 9

Pool 9 slopes appear to have changed little in the period 1957-1970 (averaging about 0.92 ft per mile), Table 5-1. In the next three years, the slopes became flatter and flatter until in November 1973 the slope was 0.84 ft per mile or 8 percent less than for preimpoundment conditions. This lessening in slope was probably due largely to reduction in downstream sediment load by Dardanelle reservoir and dredging and degradation below Dardanelle Dam. Narrowing of the river could also be a factor tending to lessen the slope.

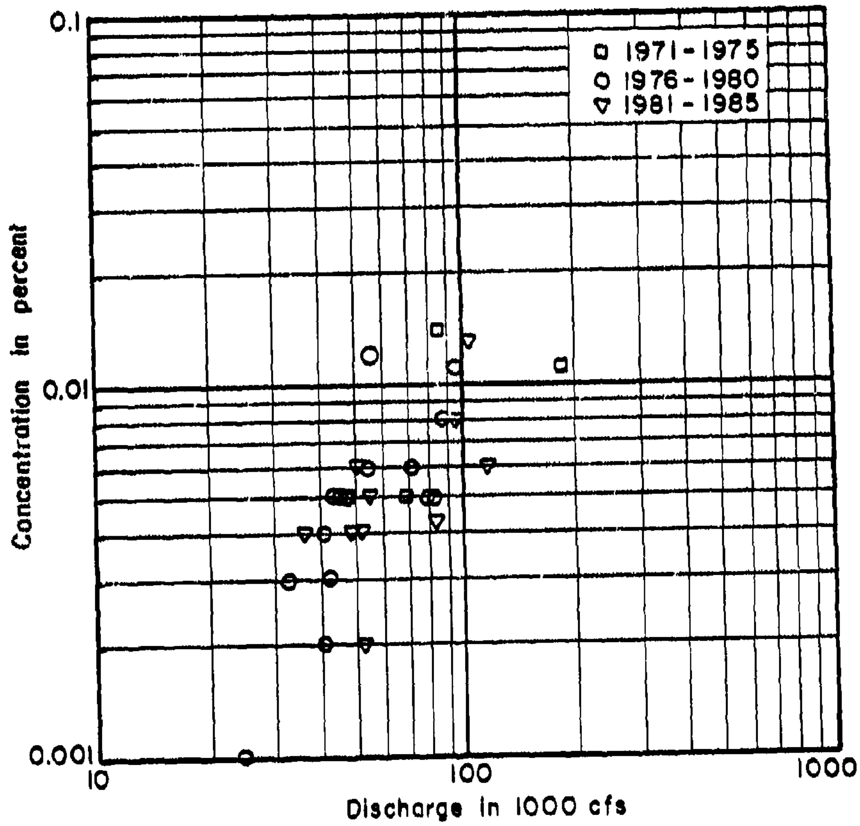
The scatter of sediment concentration data for Pool 9, Figure 5-1, does not reveal a trend, but it can be said that the concentration of about 0.01 percent by weight (100 ppm) at 100,000 cfs is only 5 or 10 percent of the concentration typical of the Arkansas River in preproject days before most of the sediment supplied from the upper watershed was trapped by the many reservoirs in the system.

By far the most, and the most important, part of the post-project sediment load in Pool 9 must be coming from the bed and banks. It would be interesting to know how much fine material escapes being trapped by Dardanelle. The two measurements with a size distribution determination at Dardanelle gage in 1976 suggest that fine material is not completely

TABLE 5-1
MEASURED WATER SURFACE SLOPES

DATE	Q (1000 cfs)	POOL 9 L[1]	POOL 9 S[2]	POOL 9 H_L[3]	POOL 8 L	POOL 8 S	POOL 8 H_L	POOL 7 L	POOL 7 S	POOL 7 H_L	POOL 6 L	POOL 6 S	POOL 6 H_L	POOL 5 L	POOL 5 S	POOL 5 H_L	POOL 4 L	POOL 4 S	POOL 4 H_L	POOL 3 L	POOL 3 S	POOL 3 H_L	POOL 2 L	POOL 2 S	POOL 2 H_L
PREIMPOUNDMENT CONDITIONS																									
4/06/57	288.0	26.6	.957	--	21.0	.760	--	30.5	.734	--	18.8	.793	--	30.1	.631	--	27.05	--	--						
4/27/57	255.0																			5.6	.750	--	38.6	.583	--
4/28/57	240.0				21.0	.614	--																		
4/29/57	270.0							30.5	.751	--	18.8	.787	--												
5/24/57	300.0																			15.8	.804	--	38.6	.624	--
5/31/57	460.0	26.6	.880	--	21.0	.905	--	30.5	.781	--															
6/01/57	460.0										18.8	.851	--	30.1	.587										
6/01/57	480.0																26.05	.763	--	15.4	.724	--	38.6	.617	--
3/14/58	105.0				21.0	.700																			
7/26/59	120.8													24.5	.820	--									
10/10/59	238.0																						38.6	.609	--
4/09/61	105.0													24.5	.868	--									
4/19/61	78.0																			15.8	.876	--			
7/20/61	157.0																26.05	.726	--						
1/04/62	38.0																			15.8	.807	--			
6/15/62	95.0													24.5	.804	--	26.05	.710	--	15.4	.665	--			
9/26/62	49.0										18.8	.737	--												
9/26/62	45.0													24.5	.849	--									
10/01/62	46.0																26.05	.726	--						
10/01/62	40.0																			15.8	.595	--			
5/02/63	55.0	26.6	.902	--	21.0	.757	--	30.5	.774	--															
5/03/63	55.5										18.8	.766	--												
5/04/63	46.5																						33.2	.819	--
4/07/64	106.0	26.6	.955	--	21.0	.710	--	30.5	.734	--	?														
7/12/67	71.5				21.0	.738	--	30.5	.807	--	17.3	.832	--				20.3	.911	--	15.4	.714	--	33.2	.861	--
3/24/68	220.0				21.0	.767	--				17.3	.890	--												
3/24/68	200.0													21.5	.936	--	20.3	.784	--	15.4	.664	--	33.2	.738	--
AVERAGE SLOPE			.916			.744			.752			.809			.782			.770			.705			.693	
PROJECT CONDITIONS																									
4/27/70	234.0																								
4/27/70	255.0				21.0	.752	0.55	30.5	.697	1.2															
4/28/70	225.0													21.5	1.02	1.6									
4/29/70	225.0																20.3	.892	0.1						
4/30/70	225.0																			15.8	.747	0.4			
4/29/70	132.0																								
4/29/70	139.0				21.0	.840	0.6																		
12/12/71	380.0	26.6	.865	0.3																					
12/12/71	250.0				21.0	.845	0.9	30.5	.705	1.0															
12/12/71	233.1													21.6	.758	0.9									
12/12/71	214.5																20.3	.867	1.0						
12/13/71	220.0																			15.8	.827	0	33.2	.848	0
4/24/73	380.0	26.6	.872	1.0	21.0	.838	0.5																		
4/24/73	370.0							30.5	.716	0															
4/26/73	350.0										17.3	1.11	0.8	21.5	.803	0.9									
4/26/73	360.0																20.3	.834	0	15.8	.848	1.1	33.2	.860	--
11/26/73	330.0	26.6	.842	1.0																					
11/27/73	320.0				21.0	.786	1.0	30.5	.711	0															
11/28/73	310.0										17.3	1.05	0	21.5	.880	0	20.3	.862	1.1						
11/29/73	300.0																			15.8	.728	1.0			
AVERAGE SLOPE			.860			.773			.708			1.08			.866			.884			.670			.651	
11/85	?		.847			.784			.798			.858			.903			.821			.687			.699	

[1] Length (mi)
[2] Slope (ft/mi)
[3] H_L at Dam (ft)



Sediment Concentration as a
Function of Discharge
Dardanelle Gage
(Pool 9)

Figure 5-1

trapped. This might explain the scatter of recent sediment loads by a factor of two or three. Unless there is an occasional supply of fine material from somewhere, that much scatter would not be expected. Individual measurements can be in error by a considerable amount because samples are small and concentration variations in the secondary-flow/large-scale turbulence of river flow are large, but hardly enough to explain all the scatter.

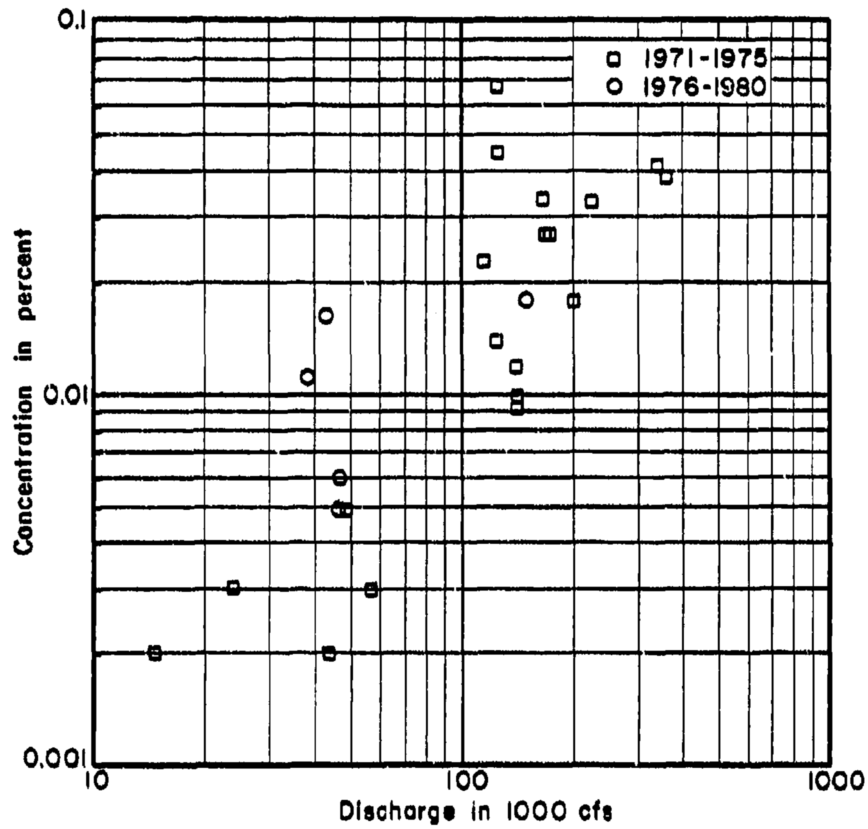
The suspended load composition in Pool 9 has no material coarser than 50-sieve size, and over 98 percent is finer than 200-sieve size. This is very fine material and is compatible with an essentially degraded stream downstream of a reservoir.

5-3 POOL 8

The slope in Pool 8 is less than through Pool 9, 0.74 and 0.77 ft per mile for preproject and post-project conditions, respectively. Although there has been essentially no change in slope, there has been considerable variation from flood to flood, Table 5-1.

Sediment concentration measurements are available only for the 1970's and do not show any trend with time, but do show scatter by a factor of five, twice that at Pool 9, Figure 5-2.

Concentration in Pool 8 is perhaps 25 percent higher than in Pool 9, and composition of the sediment load is quite different. In different floods, material is as fine as 90 percent passing the 200 sieve with 30 sieve the maximum size, and as coarse as only 15 percent passing the 200



Sediment Concentration as a
Function of Discharge
Lock and Dam 9 Tailwater
(Pool 8)

Figure 5-2

sieve and 4 sieve the maximum size. All Pool 8 samples are definitely coarser than the Pool 9 samples.

Comparing and explaining variations in concentrations and composition in individual floods is difficult. For example, for two floods in 1975 and 1976 of about the same discharge, the coarsest suspended load had a concentration about four times the concentration of the finest composition. Something else, unknown, has to account for the situation as it cannot be explained on the basis of available information.

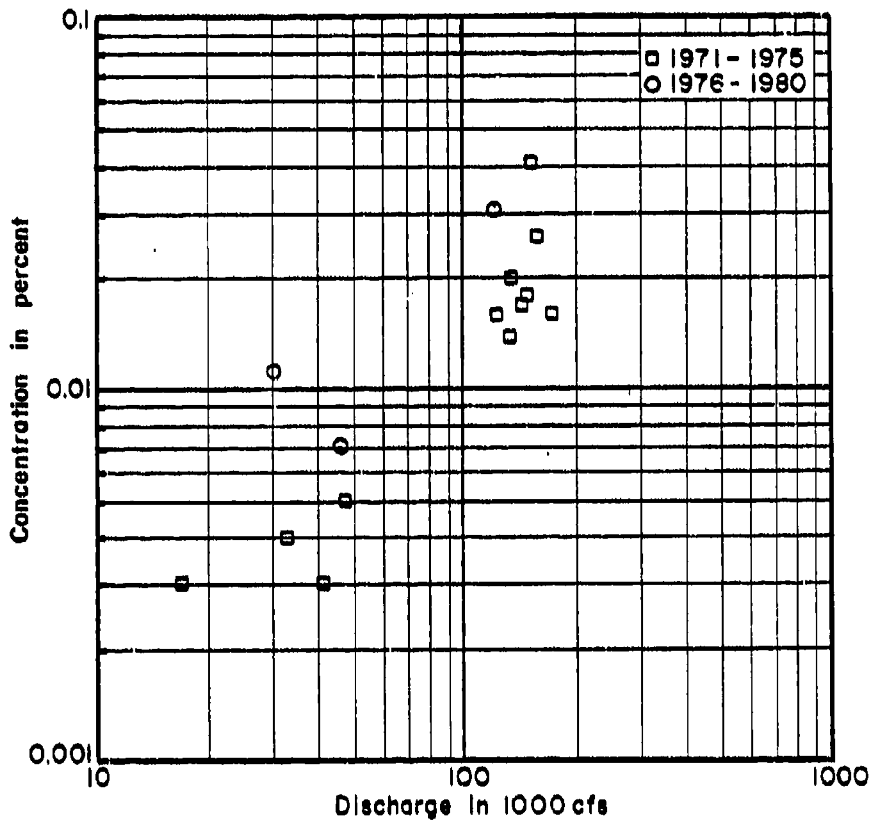
5-4 POOL 7

The preproject slope in Pool 7 was about 0.75 ft per mile, about the same as in Pool 8. By 1973 the slope had decreased to a value of 0.71 ft per mile, or by 9 percent, Table 5-1. Other samples of known composition scatter, but behave reasonably as would be expected.

The sediment concentration is essentially the same as in Pool 8, with somewhat less scatter (which may be because there is less data), Figure 5-3. The concentration vs composition comparison makes more sense, with the finest composition being a higher concentration of approximately the amount that would be expected. Other samples of known composition scatter, but behave reasonably as would be expected.

5-5 POOL 6

The preproject slope in Pool 6 (0.81 ft per mile) was greater than that of Pool 7 or Pool 8, but less than that of Pool 9, Table 5-1. Since project



Sediment Concentration as a
Function of Discharge
Lock and Dam 8 Tailwater
(Pool 7)

Figure 5 - 3

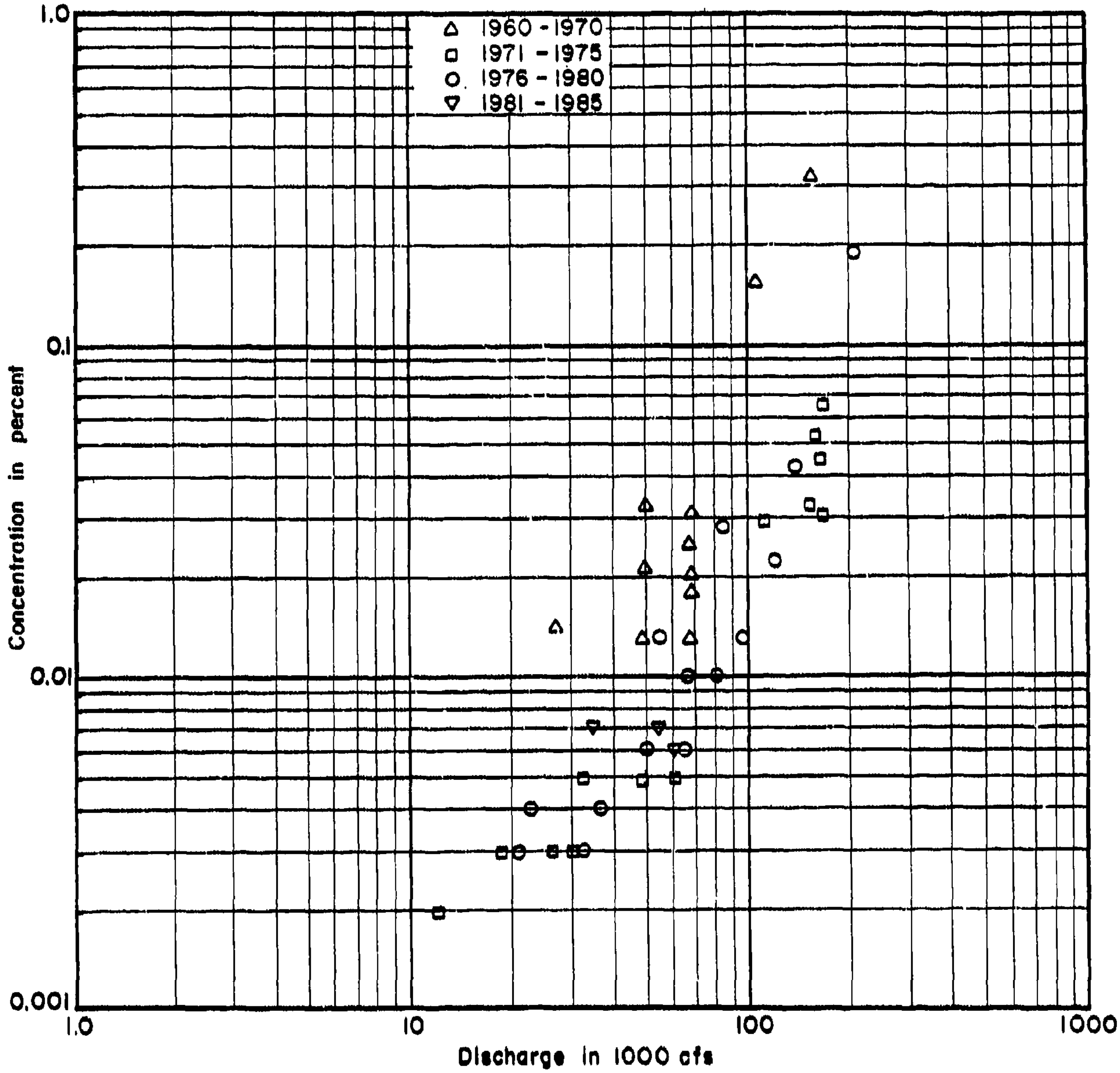
completion, the slope has increased to about 1.1 ft per mile, a 33 percent increase. The reach was shortened about 8 percent, but this change cannot explain the change in slope. The only obvious possibilities are an increase in Manning's n or an increase in sediment load. There seems to be no reason for the n value to increase in Pool 6 and not in Pools 7, 8, and 9.

The sediment load does appear to be larger in Pool 6, a concentration in the order of 0.02 percent by weight at a discharge of 100,000 cfs, Figure 5-4, compared to about 0.014 percent by weight in Pools 7 and 8 and to only 0.01 percent by weight in Pool 9. In the late 1960's, the concentration in what is now Pool 6 was five times as high, 0.1 percent by weight. The differences in sediment load may explain the greater slope in Pool 6 compared to Pools 7 and 8, but it cannot explain the increase in slope under post-project conditions compared to preproject conditions.

Not only is the concentration higher in Pool 6 than in Pool 7, but the composition appears to be finer. Finer sediment should result in higher concentration, but the question is, "Where did this greater quantity of fine sediment come from?"

Pool 6 is considered to be one of the two best pools in the lower river in terms of effectiveness of the structures in providing reliable navigable depth. Why the post-project slope is 33 percent greater than the preproject slope is a bit of a mystery.

Pool 6 seems to be an anomaly: (1) it is relatively steep, with no apparent reasons for a higher n value than other pools; (2) it has a higher



Sediment Concentration as a
Function of Discharge
Lock and Dam 7 Tailwater
(Pool 6)

Figure 5 - 4

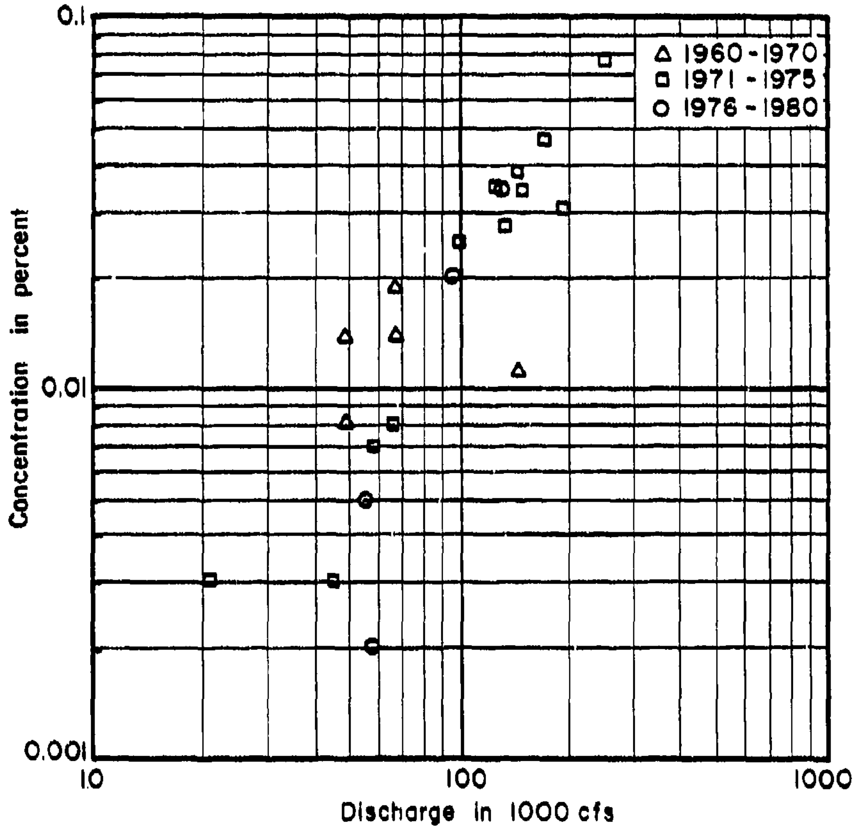
concentration of sediment than the upstream pools, with no apparent source of supply for the increase; and (3) it is apparently a well-behaved, well-designed pool with only a little dredging and some narrowing required. The anomaly may be at least partially related to relatively high bedrock levels and many bankline constrictions throughout the pool and to flow concentrations due to piers and pier protection cells for the seven bridge crossings in the vicinity of Little Rock in the middle portion of the pool.

5-6 POOL 5

Pool 5 had been shortened from 30.1 miles to 24.5 and to 21.8 miles (or a total of about 28 percent) before construction of Lock and Dam 5, but the drop in water surface through the pool remained about constant, resulting in a total slope increase of about 50 percent between 1957 and 1968. In the four years after project completion (1969 through 1973), the average slope decreased about 10 percent, Table 5-1.

The slope changes are as would be expected in Pool 5, and while degradation as a result of the cutoffs could account to some extent for the concentration being higher in Pool 5 than in Pools 7 and 8, the cutoffs were constructed in 1957 and 1962.

Pool 6 delivers a flow to Pool 5 with the same concentration as that in Pool 5, Figure 5-5.



Sediment Concentration as a
Function of Discharge
Lock and Dam 6 Tailwater
(Pool 5)

Figure 5-5

5-7 POOL 4

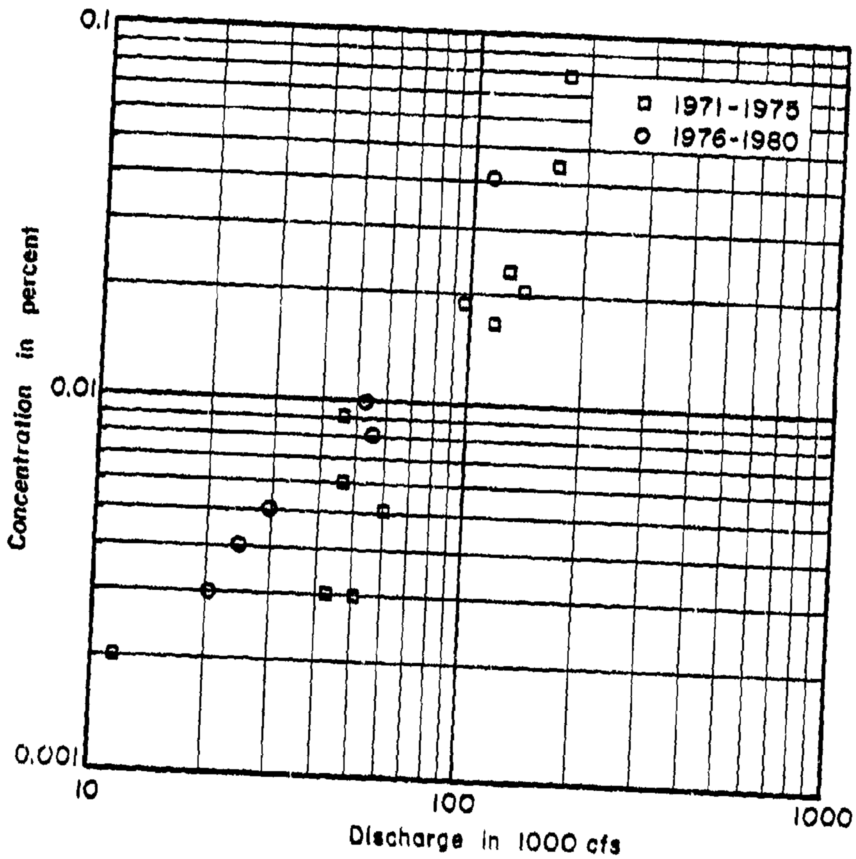
The preproject slope in Pool 4 was somewhat greater than in Pool 5, but a little less than in Pool 6, Table 5-1. The 25 percent decrease in length of the reach with the project decreased the overall drop in water surface by about 15 percent and increased the average slope about 12 percent as of 1973. The slope in 1973 was about the same as in Pools 9 and 6, but the sediment load carried in Pool 4 was almost twice that of Pool 9 and very nearly the same as in Pools 5 and 6 immediately upstream, Figure 5-6.

5-8 POOL 3

Average preproject slope in Pool 3 was 9 percent flatter than in Pool 4, Table 5-1. There was no change in reach length with the project, but the overall drop decreased 5 percent and the average slope decreased slightly, from 0.70 to 0.69 ft per mile. In 1973, Pool 3 had a slope 23 percent less than that in Pool 4, but carried the same sediment load, Figure 5-7.

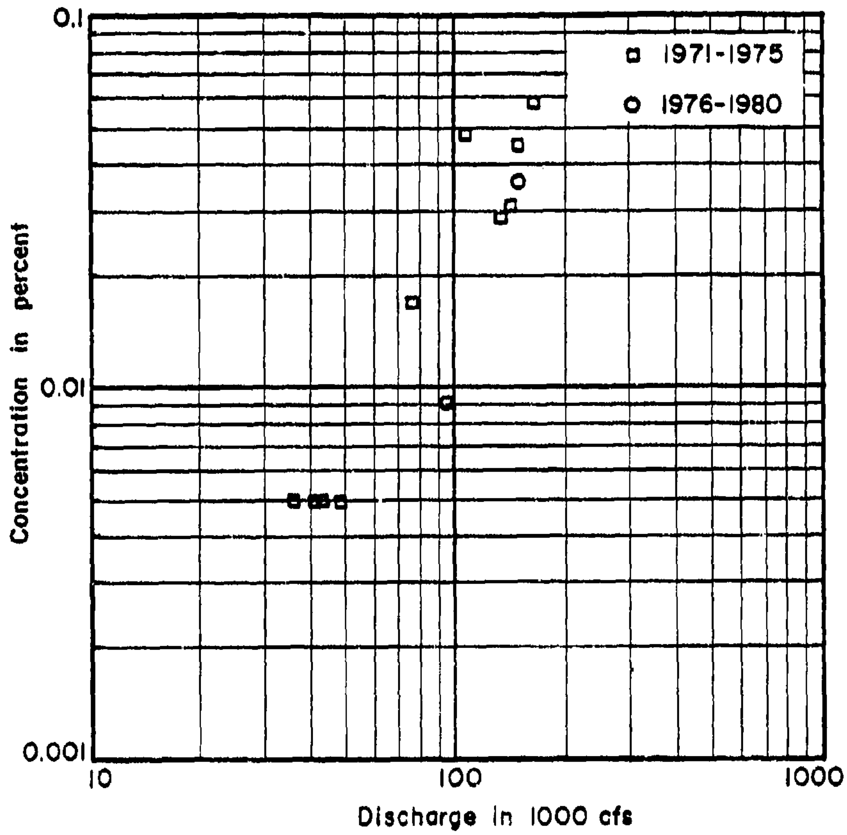
5-9 POOL 2

Under preproject conditions Pool 2 had an average slope of about 0.69 ft per mile, about the same as that in Pool 3, but 11 percent less than in Pool 4, Table 5-1. The reach was shortened 5.4 miles (14 percent) by the project, which increased the average preimpoundment slope from 0.61 to 0.81 ft per mile, or 33 percent. A few years later, in 1971-1973, the slope was considerably flatter, 0.65 ft per mile, but this probably had more to do with deposition and dredging than other factors.



Sediment Concentration as a
Function of Discharge
Lock and Dam 5 Tailwater
(Pool 4)

Figure 5 - 6



Sediment Concentration as a
Function of Discharge
Lock and Dam 4 Tailwater
(Pool 3)

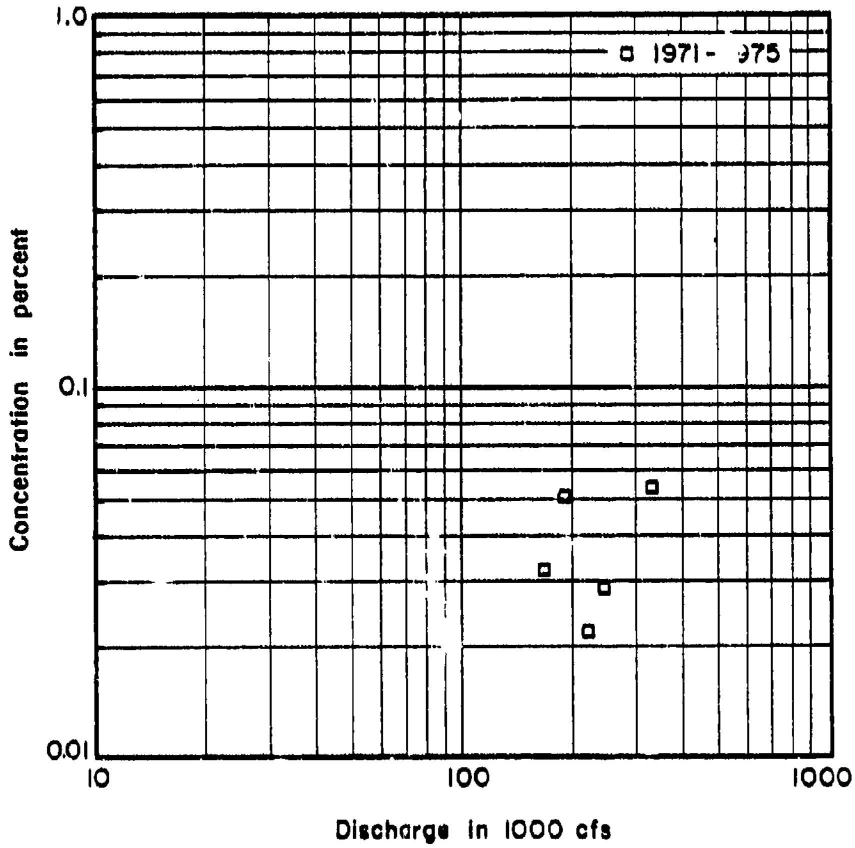
Figure 5-7

Only a few suspended sediment samples seem to have been taken in Pool 2, all in 1973, Figure 5-8. Indications are that concentrations in Pool 2 are lower than in the four pools immediately upstream. Apparently the capacity for transport in Pool 2 is half that of the pools upstream; sediment has been depositing at the head of Pool 2; and continuous dredging has been required to maintain navigable depths in the upper portion of the pool.

5-10 DISCUSSION

There are some questions about Pools 9 through 3 that cannot be answered satisfactorily. However, in general those pools are behaving adequately, and the sediment load carried is much reduced from that of free-flowing, preproject conditions. The load carried through Pool 9 downstream of Dardanelle Dam is the smallest; the next two pools downstream carry loads that are 30 to 40 percent higher, respectively; and the next four pools carry a load that is twice that of Pool 9. The additional load must come largely from the bed and banks of the pools themselves because tributary inflow downstream of Dardanelle Dam is relatively minor and mostly into Pool 9. Available data do not include data for recent years (after 1981), and it would be interesting to know what conditions are at present (1986).

The differences in sediment load are as would be expected because of the reservoirs upstream of Dardanelle Dam. As the sediment load decreased



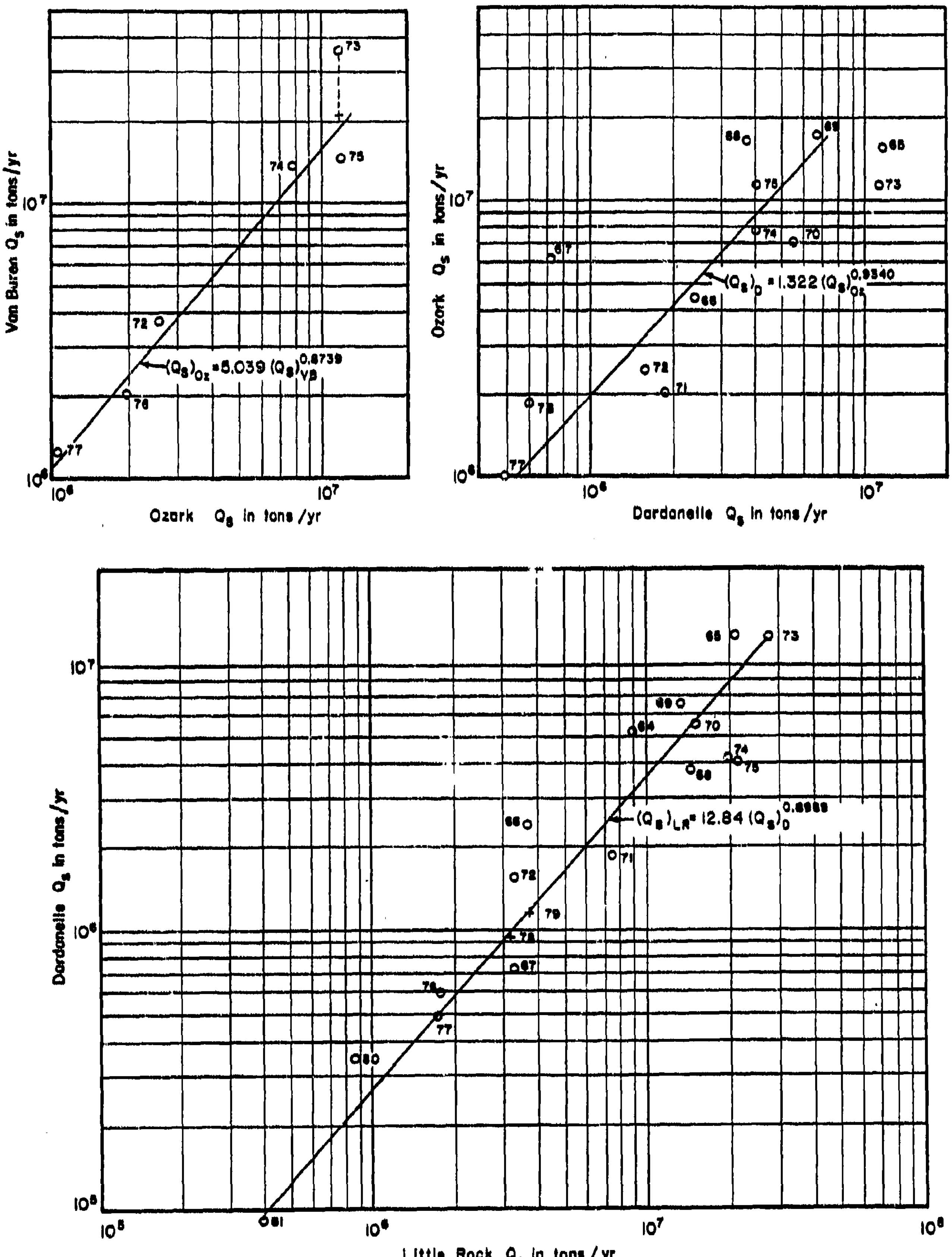
Sediment Concentration as a
Function of Discharge
Lock and Dam 3 Tailwater
(Pool 2)

Figure 5 - 8

and the river through this set of pools approaches equilibrium state, the need for dredging has decreased and has almost ceased.

Figure 5-9 and Table 5-2 illustrate the change in suspended sediment load of the lower Arkansas River due to deposition in Ozark and Dardanelle Reservoirs and the pick up in sediment in the river between Dardanelle Dam and Little Rock. The sediment load and the fraction deposited in the reservoirs are affected by the annual discharge, variation in discharge through the water year, and reservoir operation. In the period 1972-1977, deposition in Ozark ranged from 12 percent to 34 percent (averaging about 25 percent) of the incoming sediment for loads of 10^6 to 10^7 tons per year. In Dardanelle in the period 1965-1977 deposition was about 50 percent of the incoming load, with only a slight increase as the sediment load increased from 10^6 to 10^7 tons per year. For reservoirs and reservoir operation such as these, this is the behavior to be expected.

In the period 1964-1981, the sediment load between Dardanelle Dam and Little Rock increased by a factor of about three; more at lesser discharges, less at higher discharges. This variation of transport with discharge is contrary to what would be expected, especially with the low-head pools (and maintenance dredging) in the reach. However, this secondary effect may be unreal and a result of random differences and scatter in behavior for individual years. More important is the fact that, on the average, during a water year the sediment load picked up from the bed and banks in the Dardanelle-Little Rock reach was twice that supplied by releases from



Sediment Transport
Relationships

Figure 5-9

Table 5-2
Measured Annual Suspended Sediment Load and Little Rock Discharge

Water Year	Suspended Sediment Load (million tons/yr) Van Buren	Ozark	Dardanelle	Little Rock	Little Rock Discharge Annual Flow (million ac ft)	Peak Avg Daily Flow (1000 cfs)
1955				26.1	13.9	126
1956				9.1	8.1	100
1957				114.9	51.5	458
1958				75.3	35.7	184
1959				35.5	20.9	148
1960				110.5	45.3	348
1961				95.2	39.1	285
1962				46.1	31.0	161
1963[1]				16.9	12.4	63
1964			5.2	9.1	9.4	102
1965		15.6	11.7	21.6	21.0	127
1966		4.4	2.5	3.7	14.1	145
1967		6.2	0.7	3.3	12.6	86
1968		16.4	3.8	14.9	34.6	200
1969		17.1	6.8	13.8	39.3	176
1970		7.1	5.5	15.3	25.5	215
1971		2.0	1.9	7.6	19.4	156
1972	3.8	2.5	1.6	3.4	17.7	237
1973	20.2	11.4	11.3	28.6	64.4	322
1974	13.6	7.7	4.2	20.4	55.7	301
1975	14.4	11.4	4.1	21.9	56.0	236
1976	2.0	1.9	0.6	1.8	18.2	144
1977	1.3	1.0	0.5	1.7	16.4	203
1978	2.4		1.0	3.2	24.9	149
1979	2.3		1.2	3.8	28.8	173
1980	1.2		0.3	0.9	17.0	75
1981	0.3		0.1	0.4	9.3	70
1982					32.3	173
1983					34.1	271
1984					28.5	165
1985					53.0	218
Average	7.8	8.1	3.5	58.8[2] 9.7[3]	28.7[2] 28.7[4] 28.7[5] 29.0[6]	208.1[2] 179.3[4] 187.8[5]

(1) Dardanelle Dam closed in 1963
(2) Average for years 1955-1963
(3) Average for years 1964-1981
(4) Average for years 1964-1985
(5) Average for years 1955-1985
(6) Average for years 1928-1975

Dardanelle Reservoir. Such action should be degrading the reach, and in the last two years of record (1980 and 1981), there does seem to be a marked dropoff in the sediment load at Little Rock. These were quite low flow years, but the concentration seems to be half of what it was in earlier post-project years of about the same annual discharge.

Overall the water surface slopes for open-river conditions decrease in the downstream direction below Dardanelle Dam, which is normal, but there are deviations from this overall decrease in slope that cannot be explained by available data.

After data on the water-surface profile of the November 1985 flood became available late in the study, they were added to Figures 2-1 through 2-8. That flood was sizable (in the order of 220,000 cfs), but not an extremely large flood. In general, the 1985 water-surface profile was parallel to that of other floods, and as a result did not give any reason to change the observations made in this chapter. The 1985 flood confirmed the fact that the slope of the Arkansas River has not flattened as much as could have been expected. However, it should be emphasized that any questions about slope are more of academic than practical interest. The slope is not an issue in performance of the project.

Pool 2 is the only pool that continues to be troublesome with respect to deposition and the need for maintenance dredging. It can be affected by backwater from the Mississippi River; it is the longest pool downstream of Dardanelle; it appears to have operated under open-river conditions very

infrequently (in 3 of the 22 years following closure of Dardanelle Dam); and it is at the downstream end of the system where the longest time would be required to finally achieve a satisfactory equilibrium state. Perhaps these are the basic reasons it has been, and still is, troublesome.

CHAPTER 6 - OTHER INFLUENCING FACTORS

6-1 GENERAL

This current study indicates that in addition to changed discharges and sediment loads under project conditions, there are significant differences in characteristics of the eight low-lift navigation pools downstream of Dardanelle Dam that appear to be important factors in determining whether or not the stabilization and contraction works at the heads of the pools have been effective in establishing channels of adequate and reliable depths for navigation. Pool characteristics described in Chapter 2 include reference to potential effects of storage capacity frequency of open-river flow conditions, and height of stabilization structures on dredging requirements. This chapter discusses in more detail possible relationships between these factors and maintenance dredging.

6-2 POOL LEVEL WITH RESPECT TO CRP AT HEAD OF POOLS

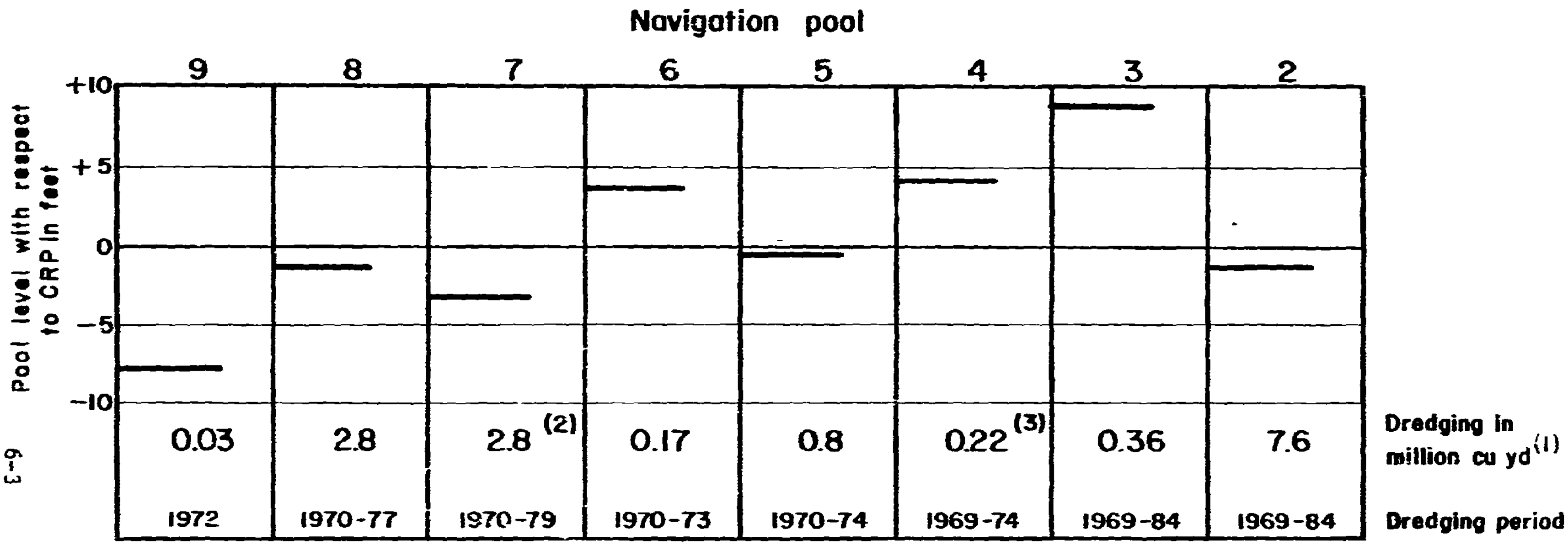
Self-maintaining navigation depths depend on the width of channels. Given a discharge and a sediment load (and a roughness value such as the Manning n), there is some width that will result in the desired depth of flow. If a pool level is set so low that conditions at the head of the pool are those of an open river, this width is the width that must be established by stabilization and contraction structures. In the early years of a project, if the sediment load is greater than it will be in later years, the depth of flow for a given width and discharge will be less than desired and

less than will be achieved in later years for the same flow and width when the sediment load is reduced.

If a pool level is set higher, greater depths of flow at the head of the pool will be provided initially, but they will not be equilibrium depths. There will be deposition at the head of the pool, gradually reducing the depth. If the time that elapses between floods is short, deposition may not be a problem as deposits that occur will be removed in the next flood.

When examining sequential operating conditions alternating between pooled low-water flow and open-river high-water flow with spillway gates fully open, the relative elevations of the stream bed under open-river conditions (or the construction reference plane generally parallel to the bed) and the water surface under pooled conditions are the important considerations. For a given discharge and sediment load, there is an equilibrium state, including the stream bed profile. As floods recede and spillway gates control releases, the pooled condition is then superimposed on the state left behind by open-river flood flow. In general, a higher pool elevation should require less maintenance dredging. Less maintenance dredging can also be achieved by narrowing the channel so that the flood flow leaves a lower bed profile behind because of both long-contraction scour and the lesser slope in the contracted reach.

Pool elevations <u>at the heads</u> of Pools 9 through 2 are shown schematically with respect to the construction reference plane in Figure 6-1



Notes:

(1) Maintenance dredging at head of pool.

(2) 0.34 mil. cu. yd. dredged in 1982−84.

(3) 0.01 mil. cu. yd. in 1983.

Pool Level with Respect to CRP at Head of Pools

Figure 6−1

together with notation of historical maintenance dredging at the heads of the pools. It will be noted that:

1. Pool 9 is set the lowest of the pools with respect to CRP and has required essentially no dredging; however, the fact that maintenance dredging was not required is probably due to extensive dredging done as a part of project construction and the decrease in sediment content of Dardanelle Dam releases.

2. Pools 8 and 7 are set higher than Pool 9, but below CRP. Although there was considerable initial dredging done at the heads of both pools as a part of project construction, both required extensive dredging in the early years of operation. Relatively modest dredging has been required since the late 1970's, as the pools apparently approached equilibrium under project conditions.

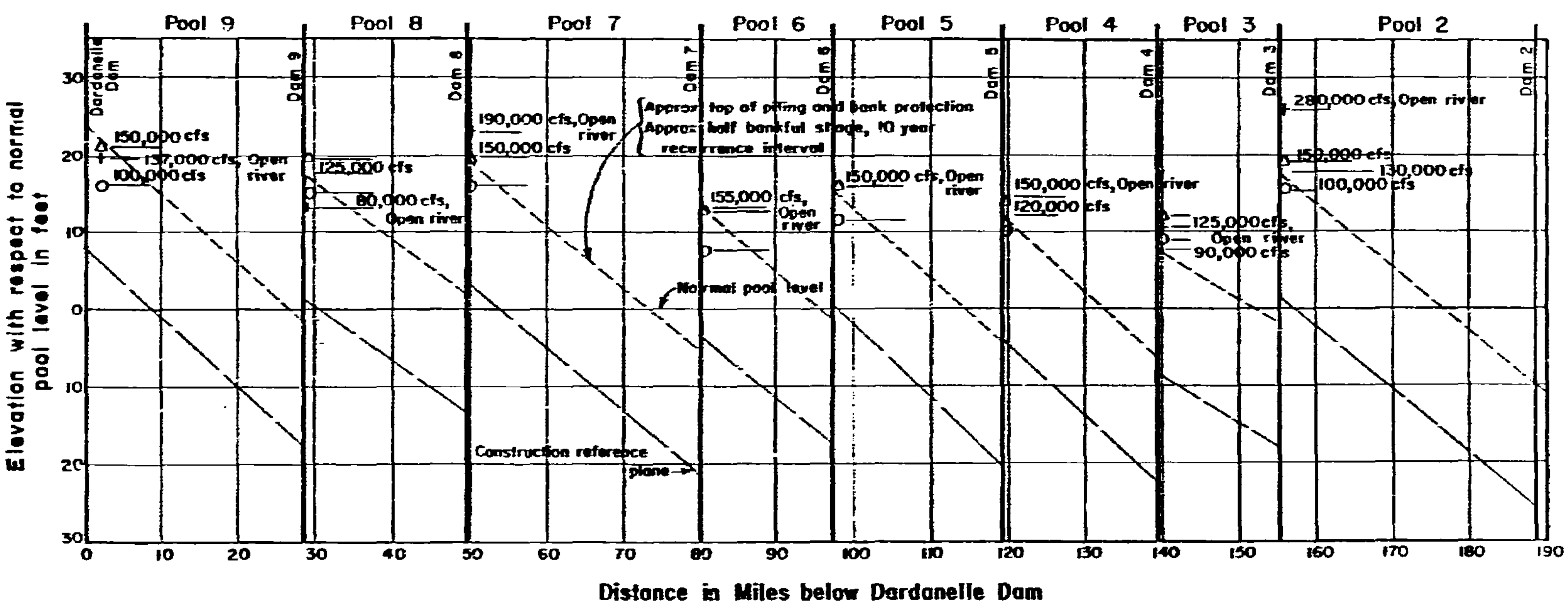
3. Pool 6 is set high with respect to CRP, but dredging requirements were modest and occurred in early years of operation, prior to 1974.

4. Pool 5 is set lower than Pool 6, at about CRP elevation, and required more dredging in the early years of operation, but no dredging after 1974.

5. Pool 4 is set high with respect to CRP, and essentially no dredging has been required since 1974.

6. Pool 3 is set the highest of the pools, and while dredging volumes have been modest some work has been required in most years.

7. Pool 2 level is set about the same with respect to CRP as Pools 8 and 5, but about 10 ft lower than Pool 3 immediately upstream. Dredging has been required to remove large volumes of deposited material almost every year. The rectified channel design at the head of Pool 2 narrowed the reach to about 800 to 1,000 ft, and it does not appear that extensive further narrowing could be employed to increase depths without raising high-water elevations.



Note: As piling deteriorates,
stone is added to pile structures
to 10 ft above CRP.

Elevation of structures and
flow lines with respect to CRP

Figure 6-2

Analysis of these data indicates that for the range of pool elevations used on the lower Arkansas River (from 8.5 ft above to 8 ft below CRP), the pool elevation per se at the heads of the pools probably had little effect on dredging requirements at the heads of the pools. (In this sense, pool level at the heads of pools is considered an isolated factor, separate from pool storage capacity and average pool depth which are more functions of length of pool.) It appears that other influencing factors, such as submergence of structures under open-river flow conditions, pool characteristics, and frequency of open-river flows, are probably more important, as discussed later in this chapter.

6-3 POOL LEVELS, STRUCTURE HEIGHT, AND FLOW LINES

The relationships between normal pool levels, approximate structure height, and elevations at heads of pools for discharges of 100,000 cfs, 150,000 cfs, and the approximate flow at which open-river conditions begin are shown schematically in Figure 6-2. Structure height refers to top of upper bank paving and to top of piling in longitudinal pile structures and at the riverward end of spur dikes; top of piling in spur dikes is frequently higher at the bankline than at the river end. It should be noted that as piling deteriorates over time and additional stone is added to pile structures, top of stone is at about 10 ft above CRP (or 3 to 6 ft lower than the intial top of piling indicated on Figure 6-2).

Top of structures was set at elevations corresponding to approximately half bankfull stage, a flow equaled or exceeded about ten percent of the

time. It is evident that control structures in Pools 2 and 7 are much lower with respect ot flow lines when open-river conditions begin than are structures in other pools and that stone structures would be submerged in the order of 15 and 10 ft, respectively, in Pools 2 and 7.

The data in Figure 6-2 indicate that top of stone is such that structures probably are effective in controlling the flow configuration up to a discharge of about 130,000 cfs through Pool 9; 70,000 cfs through Pool 8; 100,000 cfs through Pools 7, 6, and 5; 70,000 cfs through Pool 4; 60,000 cfs through Pool 3; and 85,000 cfs through Pool 2. It follows that structures in Pool 3 are less effective in directing flows than are structures in other pools and that more upper bank scour can be expected in Pool 3 than in other pools (thus increasing the sediment load into Pool 2) due both to the low elevation of top of bank protection and to poor direction and control of moderate and high flows.

As noted, the top of rock in structures in Pools 7 and 2 is significantly below flow levels when spillway gates are fully open (open-river conditions) on rising stage when water surface slopes can be expected to be the steepest and conditions favorable for sediment transport. Structures in those pools can, therefore, be expected to be less effective in confining open-river flows to the rectified channel and in flushing sediment deposits through those pools than structures in other pools. When open-river conditions are reached in Pools 7, 4, and 3, the top of pile structures and upper bank paving would be submerged about 5, 3, and 2 ft,

respectively; in Pool 2 they would be submerged about 10 ft. The 1981 aerial mosaics of the navigation project indicate that dike fields in Pool 2 downstream from Lock and Dam 3 generally have less vegetation growing on them than other dike fields in the system. This may in part be due to maintenance dredge material deposition, but it also could indicate that much of this material is being washed into the navigation channel during flood flows.

The adverse effects of submergence in Pool 3 are partially offset by the fact that open-river conditions prevail for longer periods and somewhat more frequently than in Pools 4 and 5 upstream, Table 6-1.

6-4 OPEN-RIVER FLOW CONDITIONS AND POOL SIZE

As discussed in Chapter 1 and Section 4-6, design studies indicated that essentially all the sediment load would be transported through the low-lift navigation pools. Material deposited when those pool levels are controlled by spillway gates was expected to be resuspended and transported downstream under open-river flow conditions on the next rise when gates would be fully open. However, Pools 7 and 2 have somewhat different characteristics than other pools downstream from Dardanelle Dam. They are the largest pools in terms of storage, the longest, and are subject to open-river flow conditions less frequently and for shorter periods than the other pools.

Minimum discharges at which spillway gates are fully open and open-river flow conditions prevail throughout the pools downstream of

Dardanelle Dam vary greatly from pool to pool, ranging from 80,000 cfs at Dam 8 to 280,000 cfs at Dam 2. These flows are discussed briefly by pool in Chapter 2 and are summarized in Table 6-1. Related data in Table 6-1 on frequency and duration of open-river flow are based on historical data at Little Rock for the 22-year period 1964-1985 following closure of Dardanelle Dam. Although that period is not as long as desirable for statistical analysis, it reflects operation of the upstream storage projects and contains a number of high- and low-flow years. The average annual discharge (28.7 million ac ft) is about the same as that for the 1928-1975 period (29.0 million ac ft), Table 5-2. Peak average daily flow data at Little Rock was assumed to be roughly representative of frquency and duration of discharges at all locks and dams downstream of Dardanelle Dam.

As noted previously, open-river conditions in Pool 7 have a recurrence interval of about 2.4 years and occur about one percent of the time, on the average. In Pool 2 the recurrence interval is about 7.3 years and duration about 0.1 percent of the time, on the average. Sediment transport through Pools 7 and 2 can be expected to be much less efficient than through the other pools because: (1) structures in Pools 7 and 2 probably do not effectively confine flows to the narrowed rectified channel because of submergence at the time open-river conditions are reached when water surface slopes are steep and sediment transport rates are high, and (2) open-river conditions occur less frequently (particularly in Pool 2) than in the other pools where gates are fully open almost every year, Table 6-1.

TABLE 6-1

Frequency and Duration of Open-River Flow Events[1]

Pool	Approximate Open-River Discharge (cfs)	Flow Duration % Time (approx) Design Studies	Experienced 1964-85[1]	High Flow Years[2] Number	%	Return Interval (years)[3]	Avg Number of Events in High Flow Years[4]	Avg Number of Days Per Event[5]
9	137,000	6	5.0	17	77	1.3	3.2	6.0[6]
8	80,000	15	14.6	20	91	1.1	4.9	11.0[7]
7	190-200,000	2	1.1	9	41	2.4	1.9	3.8[8]
6	155,000	6	3.0	13	59	1.6	2.7	4.9[9]
5	145-150,000	6	4.2	16	73	1.4	2.4	6.9[10]
4	145-150,000	6	4.2	16	73	1.4	2.4	6.9[10]
3	125,000	8	6.8	18	82	1.3	2.9	8.9[11]
2	270-280,000	--	0.1	3	14	7.4	1.0	2.7

(1) Based on Little Rock mean daily discharge in the 22-yr period 1964-1985 following closure of Dardanelle Dam.

(2) Number of years in 22-yr period in which peak mean daily discharge at Little Rock exceeded open-river discharge.

(3) Based on 22-yr period.

(4) Total number of flow events in which peak mean daily discharge at Little Rock exceeded open-river discharge divided by number of high-flow years with open-river conditions.

(5) Average number of days per event in which mean daily discharge at Little Rock exceeded open-river discharge.

(6) Excluding one 82-day event in 1973; 57% of all events of 3-days duration or less.

(7) Excluding one 87-day event in 1973; 31% of all events of 3-days duration or less.

(8) Excluding one 25-day event in 1973; 59% of all events of 3-days duration or less.

(9) Excluding one 67-day event in 1973; 59% of all events of 3-days duration or less.

(10) Excluding one 80-day event in 1973; 42% of all events of 3-days duration or less.

(11) Excluding one 84-day event in 1973; 42% of all events of 3-days duration or less.

As discussed in Section 4-6 it appears likely that with Pool 8 operating at open-river conditions essentially annually and for long periods of time, there is good transport of sediment through Pool 8 and into the headwaters of Pool 7. Pool 7 has 2.7 times the storage volume of Pool 8 at normal pool, is approximately 50 percent longer, and operates under open-river conditions about once in 2.4 years for about four days, on the average. Sediment inflow to Pool 7 apparently exceeds the capacity for transport through the pool.

Likewise for Pool 3, which operated under open-river conditions in 73 percent of the years in the 1964-1985 period, as discussed in Section 4-6, sediment transport through the pool has been good. It appears that the material transported into Pool 2 greatly exceeds the transport capacity through Pool 2. Pool 2 has 2.2 times the storage volume of Pool 3 at normal pool, is more than twice as long as Pool 3, and appears to have operated under open-river conditions for short periods of one to four days about one year in seven, much less frequently than Pool 3.

For both Pools 7 and 2, available data indicate that factors that distinguish those pool from other pools downstream of Dardanelle Dam are the length and storage volume of the pools at normal pool level and the less frequent occurrence of open-river flow through the pools. These characteristics are more different for Pool 2 than Pool 7, and Pool 2 has more severe problems of sediment deposition and maintenance dredging than Pool 7.

Additionally, with spillway gates at Dam 2 fully open and high Mississippi River stages, backwater from the Mississippi extends upstream into Pool 2. This is a complex flow and stage situation, and virtually no information was available relative to the effects of Mississippi River stages on conditions in Pool 2 since the project became operational. In general, Mississippi backwater would reduce water surface slopes through Pool 2 and decrease the sediment transport capacity of Arkansas River flows, contributing to deposition in Pool 2.

6-5 SPILLWAY GATE SILL ELEVATIONS

Design studies were based on setting spillway gate sill elevations for Dams 9 through 2 at the elevation of the future degraded bed. In the original plan, the sills were stepped in elevation across the channel to conform to the predicted varied bed elevation. However, only Dam 4 was constructed with a sill at two levels. Sills for the other dams are level across the channel at an average bed elevation that is generally above the thalweg level, as shown in Figures 2-1 through 2-8.

Design studies indicated that head losses through the low-lift navigation dams would be in the order of one foot at each structure for open-river conditions. Available data indicate that losses in the early 1970's were of about that magnitude. The small losses at each dam add up to about 8 ft downstream of Dardanelle, and this loss has been compensated for by the decreased slopes generally experienced and the decreased lengths due to cutoffs.

In the future, if water surface slopes flatten, the spillway sills will perhaps control the drop through the dams and the upstream bed elevations, and losses through the dams in the future may be somewhat greater than at present.

CHAPTER 7 - EVALUATION OF DESIGN CRITERIA

7-1 DESIGN CRITERIA

Design criteria for stablization and rectification work on the Arkansas River can be summarized as follows:

1. Rectified alignment

- Follow easy bends of alternate curvature with as much sinuosity as possible.
- Follow the natural configuration of the river to maximum extent practicable to minimize costs.
- Avoid long straight reaches and reaches of little curvature.
- Cut off short-radius bends to reduce revetment maintenance and improve navigation conditions.
- Use smooth transitions from bends to tangents in crossings.
- Base rectified cross sections on characteristics of naturally stable reaches.

2. Crossings

- Crossing length to be 2 to 4 times channel width (1,000 to 15,000 ft).
- Longitudinal structures on one bank only to permit future additional contraction if required.

3. Bends

- Revetments to have no irregularities or false points that might induce local scour.
- Use spur dikes on convex bank in bends to contract channel to permit future additional contraction if required.

4. Structures

- Use the cheapest type structure that is appropriate for a site.

- Work from one fixed point downstream to another to maintain control of the reach.

- Protect upper banks to about half bankfull stage (stage equaled or exceeded about ten percent of the time).

7-2 CRITIQUE OF DESIGN CRITERIA

In general, structures on the lower Arkansas River designed in conformance with the above criteria have performed remarkably well. However, the criteria were not always strictly adhered to because of site-specific conditions. General comments on design criteria are as follows:

1. To minimize costs and accomodate rock escarpments, less than desired sinuosity was provided in some reaches, particularly in Pools 9, 8, 7, and 6. The adopted alignment has generally provided adequate navigable depths in the heads of the pools probably in large part due to reduction in sediment load with Dardanelle Reservoir in operation.

2. Additional contraction has been provided in some crossings and on the convex banks of some bends. Such modifications generally could be accommodated in the original layout. Crossing widths of 700 to 1,000 ft appear to be more appropriate than the wider crossing widths of the original design. Shorter crossings have given better performance than longer crossings that sometimes greatly exceeded design criteria for length because of specific local conditions.

3. Structure heights were generally adequate except in Pools 7 and 2. For effective control on a canalized river, the stage corresponding to establishment of open-river flow conditions probably should be at about the elevation of top of control structures.

4. Top of bank protection in Pool 3 corresponds to a discharge prevailing about 13 percent of the time, and upper bank erosion is likely one source of the sediment deposited in Pool 2. Protection probably should be carried to a flow line having a duration of from 5 to 10 percent of time, as in the other pools, to minimize upper bank erosion under open-river conditions.

7-3 EFFECT OF POOL CHARACTERISTICS ON STRUCTURE PERFORMANCE

Bank stabilization and channel rectification work was laid out and much of it was constructed before sites and pool elevations for the low-lift navigation structures downstream of Dardanelle were finally established. Analysis of maintenance dredging records and pool characteristics indicates that long deep pools, in particular Pools 2 and 7, probably act as sediment traps and that unless open-river flow conditions occur frequently (probably at least annually) it is unlikely that the channel can be contracted sufficiently without raising flood heights to ensure that all sediment deposits will be scoured out of those pools during open-river flow.

This study indicates that maintenance dredging volumes in Pools 2 and 7 probably are more closely related to: (1) storage capacity of the pool; (2) detention time in the pool; and (3) the infrequent and short duration of periods when spillway gates are fully open and open-river flow conditions prevail through the pool than to design of the stabilization and recitification structures. The degree of submergence of structure in Pools 2 and 7 is probably a contributing factor. To more fully understand the deposition problems in Pool 2, additional study and field data are needed.

CHAPTER 8 - CONCLUSIONS

Conclusions of this study are as follows:

1. Stabilization and rectification structures constructed on the Arkansas River downstream of Dardanelle Dam have performed exceedingly well with respect to providing design depths for navigation, although:

   a. Additional contraction of the channel was required in some reaches; the possibility of the need for additional contraction was recognized in project design because of the many uncertainties involved.

   b. Major cutoffs in the reach downstream of Dardanelle were opened and much of the stabilization and rectification work had been constructed several years before closure of the low-lift navigation dams, but significant volumes of maintenance dredging were required in most pools in the first five years of operation, 1969 - 1974.

   c. Significant volumes of material continue to be dredged from Pool 2 each year, but it is likely this problem is more directly related to characteristics of the pool and the infrequency of open-river flow conditions than to design of the stabilization and rectification works.

2. On the basis of performance of the structures in the heads of Pool 9 through 3, the design criteria used are appropriate for free-flowing river conditions and in the heads of navigation

pools where open-river flow begins at an elevation approximately the same as the top of structures in the heads of the pools and where open-river flow conditions occur at least annually.

3. A better understanding is needed of sediment transport through Pool 2 to define the basic sources and composition of deposits at the head of that pool. A more detailed study based on additional field data is needed.

4. Project design envisioned that as sediment is trapped in the upper reaches of Dardanelle Reservoir, control structures would be needed and would eventually extend essentially throughout the reservoir. It appears that a better understanding of the deposition problems in Pool 2 might lead to a more effective and more economical design for additional contraction works in Dardanelle Reservoir.

5. The performance of stabilization and rectification structures on the Arkansas River indicates that the basic criteria on which design of the structures was based are sound and probably can be applied, using discretion, to work on other similar alluvial rivers.

APPENDIX A - GLOSSARY

Advance maintenance dredging -

Dredging in excess of authorized project dimensions to allow time for sediment buildup before the 9-ft authorized depth is no longer available and maintenanace dredging must be repeated.

Average pool depth -

The ratio of storage volume at normal pool level to surface area at normal pool.

Canalization -

Conversion of a free-flowing river to a series of navigation pools by construction of locks and dams.

Construction reference plane (CRP) -

The vertical control line on the Arkansas River; a sloping plane corresponding approximately to the mean low-water profile for preproject conditions (about 10,000 cfs) and adjusted for river shortening under project conditions.

Crossing -

Relatively straight stream reach between bends of reverse curvature.

Cutoff -

A new and shorter channel across the neck of a longer, shorter-radius bend, either natural or constructed.

Dikes -

Control structures at an angle to the stream flow.

Hinged pool operation -

Lowering the water surface profile below normal pool by operation of spillway gates on the rising leg of a flood hydrograph to lower flood profiles.

Initial dredging -

Dredging to more than authorized channel dimensions at heads of pools as a part of project contruction to hasten establishment of equilibrium channel conditions.

Natural stream flow -

Historical data prior to construction of regulating reservoirs and later recorded data adjusted to eliminate the effects of reservoir regulation.

Navigation mile (N mile) -

Mileage along navigation channel established in 1972; mile zero is at mouth of White River.

Open-river conditions -

No control of river flow or water surface elevation by dams; spillway gates on low-lift navigation dams are fully open.

Pool -

The lake-like reach of a river upstream of a navigation dam.

__Revetments__ -

Structures parallel to the stream flow used to control stream alignment and/or to stabilize stream banks and prevent bank erosion.

__Wash load__ -

That part of the suspended sediment load comprised of silts and clays (diameter <0.004 mm).